Pretensioned girder frame bridge

CONTENTS

1. GENERAL / MEASUREMENT

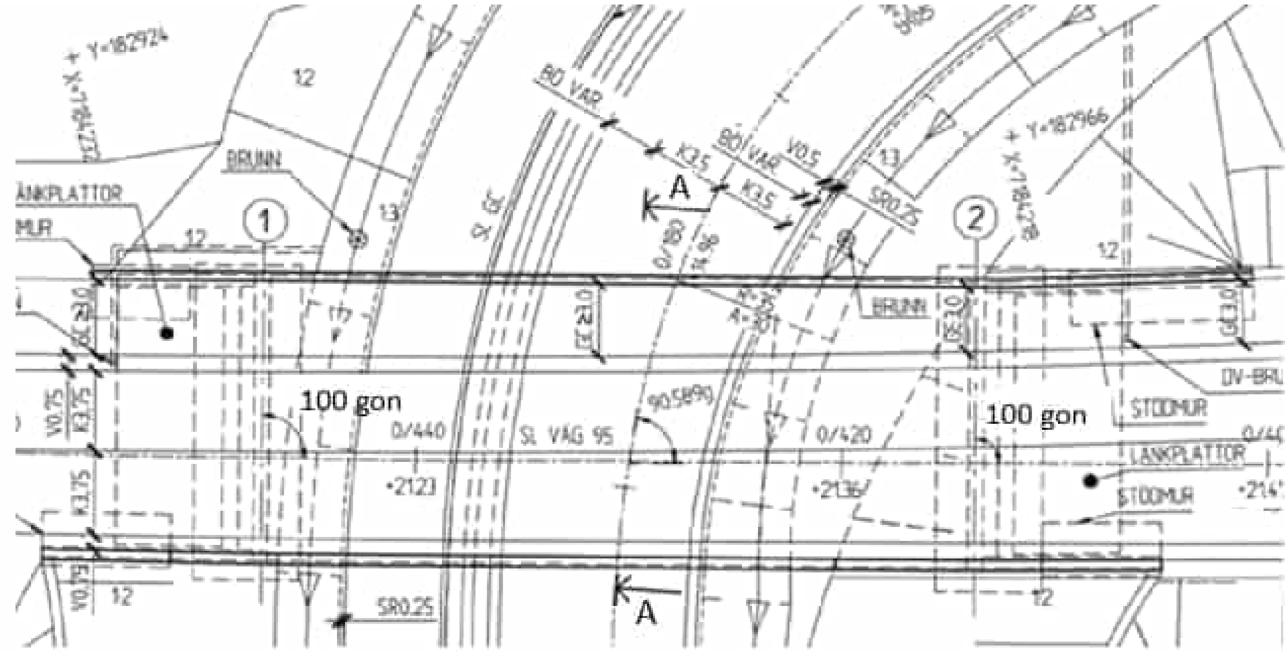
1.1 CONSTRUCTION TYPE

Beam frame constructed using reinforced concrete. Superstructure is modelled with longitudinal beams that are pretensioned.

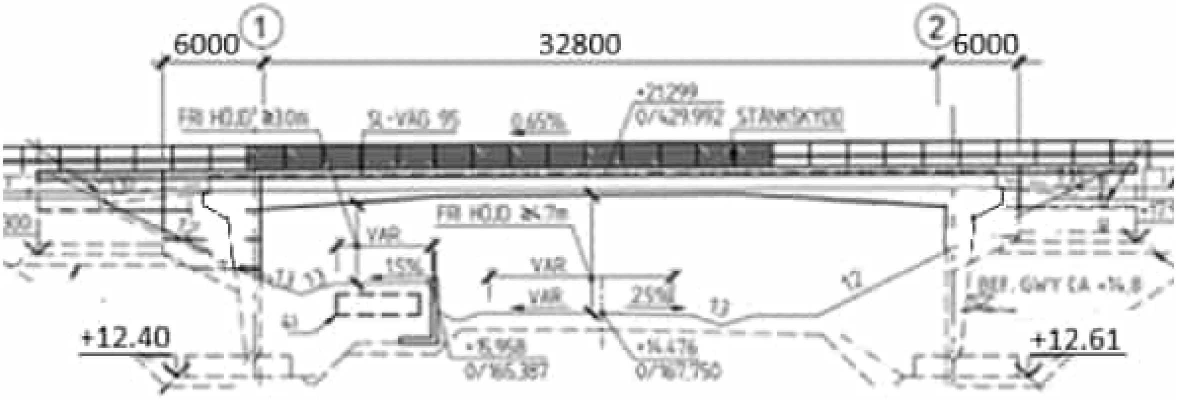
Foundation on compacted gravel.

1.2 MEASURMENT

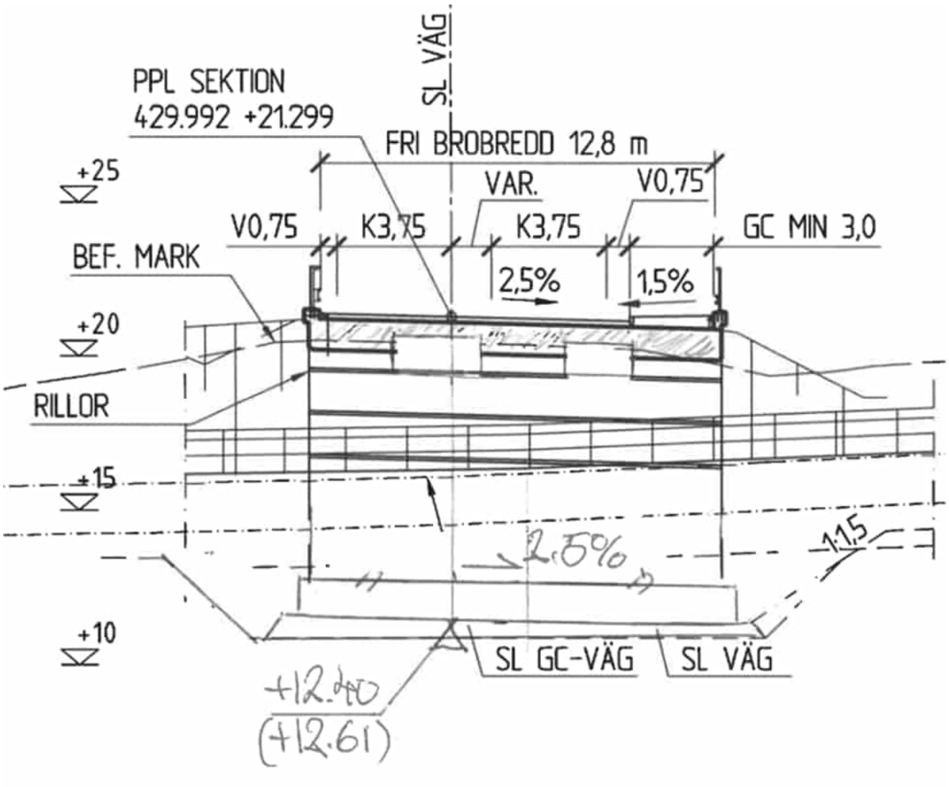
1.2.1 Overview



PLAN



ELEVATION

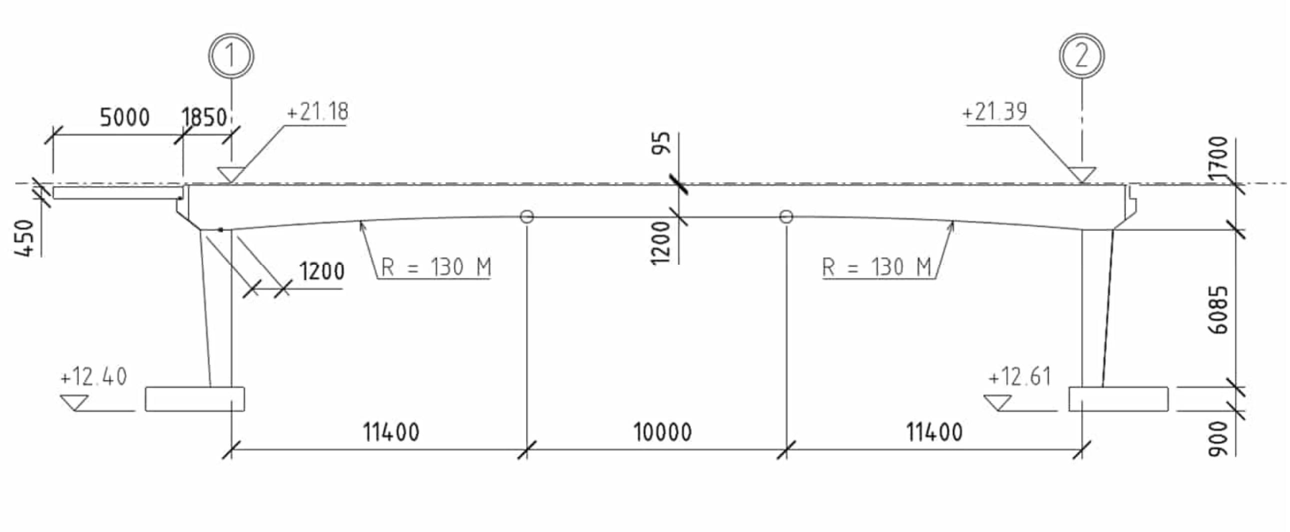


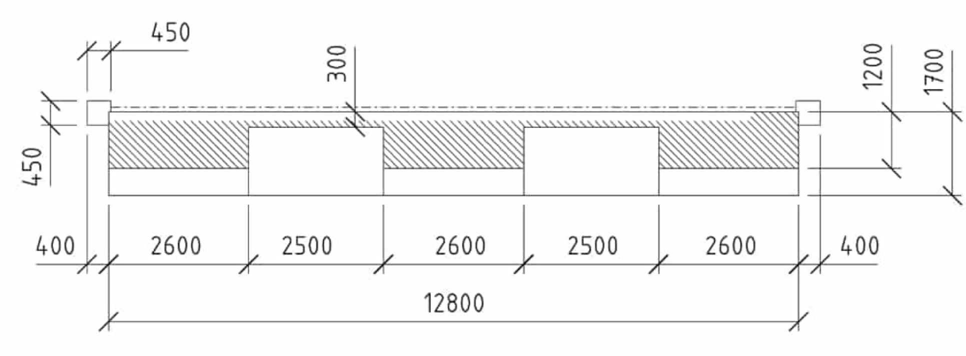
SL VÄG
PPL SEKTION
429.992 +21.299
FRI BROBREDD 12,8 m
+25
VAR.
V0,75
V0,75
K3,75
K3,75
GC MIN 3,0
BEF. MARK
2,5%
1,5%
+20
RILLOR
+15
2,5%
1:1,5
+10
SL GC-VÄG
SL VÄG
+12.40
(+12.61)

SECTION A-A

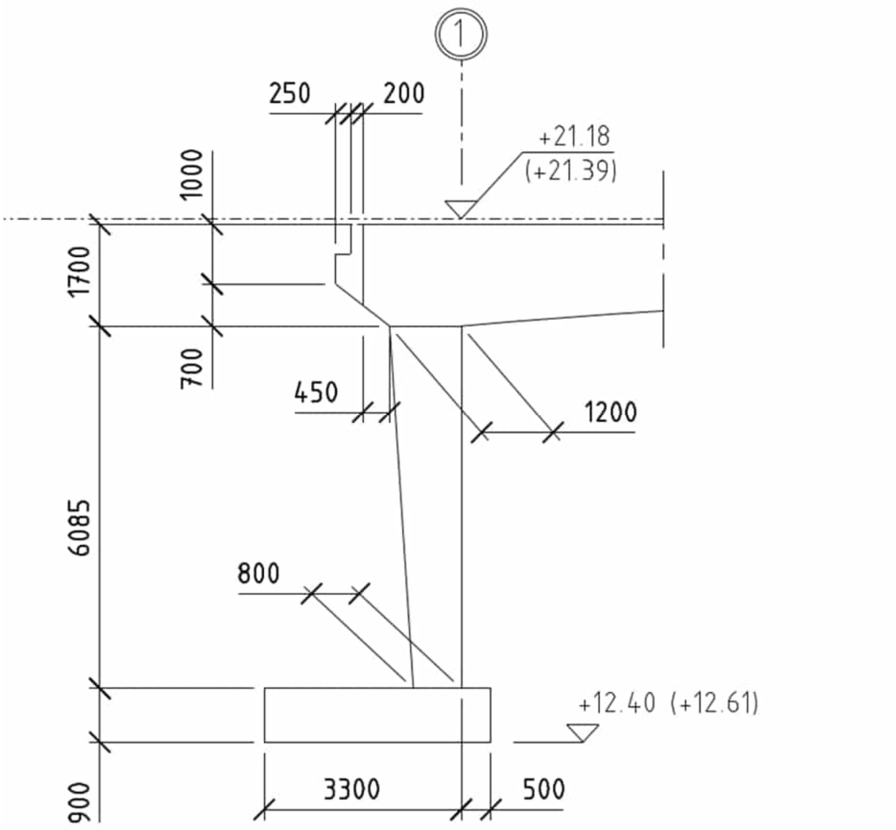
1.2.2 Detailed measurement

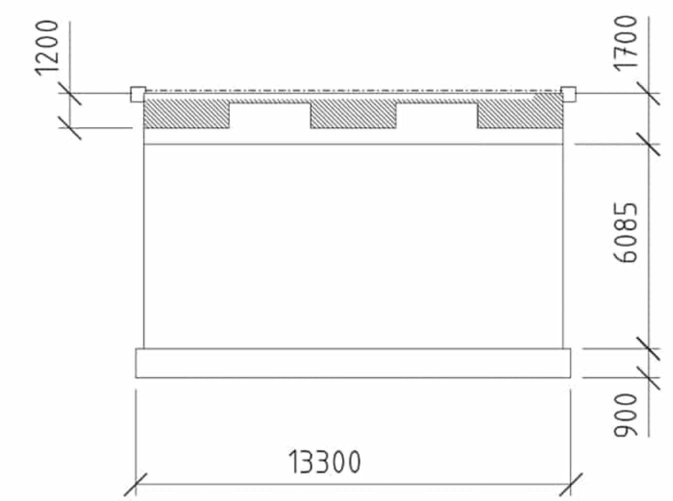
Superstructure:





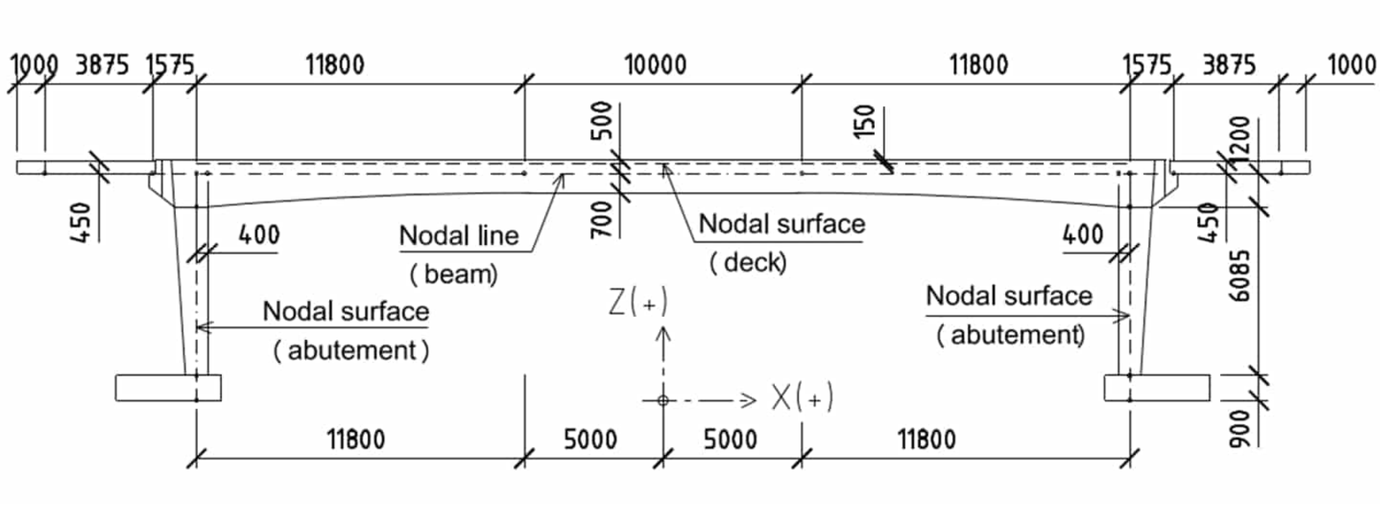
Substructure:

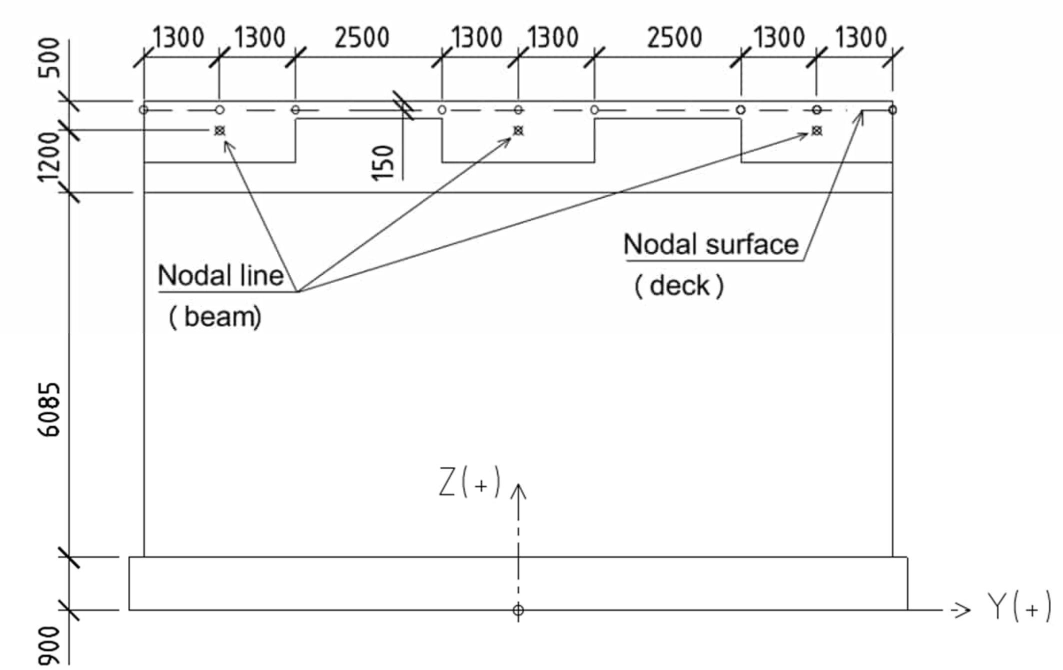




1.2.3 Measurements FEM-modell

Simplified measurements used in FEM-modell.





1.3 FOUNDATION

Both bottom slabs are founded on 0.3 m compacted gravel on rock.

1.4 CODE DOCUMENTS

Documents	Version	Name
SS-EN 1990-1997	-	Svensk Standard Eurokod 1-7
TRVINFRA-00226	2.0	KRAV, Bro och broliknande konstruktion, Allmänna krav
TRVINFRA-00227	2.0	KRAV, Bro och broliknande konstruktion, Byggande
TRVINFRA-00228	2.0	KRAV, Bro och broliknande konstruktion, Brounderhåll
TRVINFRA-00331	2.0	KRAV, Bro och broliknande konstruktion, Bärighetsberäkning
TSFS 2018:57		Transportstyrelsens föreskrifter och allmänna råd om tillämpning av eurokoder
TDOK 2013:0667	2.0	Trafikverkets tekniska krav för geokonstruktioner. TK Geo 13
TDOK 2013:0668	2.0	Trafikverkets tekniska råd för geokonstruktioner. TR Geo 13
AMA Anläggning 23		AMA, Svensk Byggtjänst
TDOK 2023:0125	2.0	TRVAMA Anläggning 23
SS 137006:2015	-	Betongkonstruktioner – Utförande – Tillämpning av SS-EN 13670:2009 i Sverige

1.5 TECHNICAL SERVICE LIFE

Technical life span 120 years (L100).

1.6 ENVIRONMENT

Pedestrian environment is assumed for the underlying pedestrical road.

Road traffic environment is assumed for the overlying traffic road.

1.7 MATERIAL

Concrete :	C35/45 & C40/50 (CEM I 42.5 N, Anläggningscement klass N)
Reinforcement :	B500B
Compacted fill :	”Förtärkningslagermaterial” according to AMA CEB.415
Backfill :	”Grovkrossad sprängsten” according to AMA CEB.524
Surfacing :	See document RKFM
Pretension:	VSL system or equivalent

1.8 GEOTECHNICAL CLASS

Geotechnical class GK2

1.9 SAFETY CLASS

Geotechnical resistance: SK 2

Bridge structure : SK 3

Retaining walls: SK 3

1.10 CONCRETE COVER AND CRACK CRITERIA

Class identification bridge components :

Bridge components	Exposure class[1.)]	Life spann	max vct_{ekv}[2.)]	ζ [3.)]
Substructure incl. linkplate:				
▫ Wingwall towards filling	XD1/XF4	L100	0.45	1.5
▫ Wingwall from filling	XD1/XF4	L100	0.45	1.5
▫ Abutemnt below ground	XC2/XF3	L100	0.50	1.0
▫ Abutement in air	XC4/XF3	L100	0.50	1.2
▫ Bottomslab in general	XC2/XF3	L100	0.50	1.0
▫ Bottomslab underside	XC2/XF3	L100	0.50	1.0
▫ Linkslab in general	XD3/XF2	L100	0.40	1.8
▫ Linkslab underside	XD3/XF2	L100	0.40	1.8
Superstructure:				
▫ Edge beam	XD3/XF4	L100	0.40	1.8
▫ Bridge deck	XD1/XF4	L100	0.40	1.5
Retaining wall:				
▫ Wall towards filling	XD1/XF4	L100	0.40	1.5
▫ Wall from filling	XC4/XF3	L100	0.40	1.2
▫ Bottomslab in general	XC2/XF3	L100	0.40	1.0
▫ Bottomslab underside	XC2/XF3	L100	0.40	1.0

Footnote:

[1.)] TRVINFRA-00227 section 5.3.2.3

[2.)] TSFS table 12.1

[3.)] TSFS table 12.3

Design parameters low corrosion sensitive reinforcement (rebars):

$c_{min.dur}$	: minimum cover with regard to environmental impact
$c_{min.b}$	: minimum cover with regard to adhesion requirements
Δc_{dev}	: execution tolerance
c_{min} = max ($c_{min.b}$; $c_{min.dur}$; 10mm)	: SS-EN 1992-1-1 eq. 4.2
c_{nom} = c_{min} + Δc_{dev}	: SS-EN 1992-1-1 eq. 4.1, noted as BM on the drawing

Construction part	$c_{min.dur}$ [1.)]	$c_{min.b}$ [2.)]	c_{min}	c_{dev} [3.)]	c_{nom}	$w_{k,till}$ [4.)]
Substructure including link slab:						
▫ Wing wall against fill	30	20	30	10	40	0.20
▫ Wing wall from fill	30	20	30	10	40	0.20
▫ Frame legs below ground	20	20	20	10	30	0.40
▫ Frame legs above ground	25	20	25	10	35	0.30
▫ Bottom slab (general)	20	20	20	10	30	0.40
▫ Underside of bottom slab	20	20	20	10	30	0.40
▫ Link slab (general)	45	20	45	10	55	0.15
▫ Underside of link slab	45	20	45	10	60[5.)]	0.15
Superstructure:						
▫ Edge beam	45	20	45	10	55	0.15
▫ Bridge deck	25	20	25	10	35	0.20
Retaining wall:						
▫ Wall against fill	25	20	25	10	35	0.20
▫ Wall from fill	25	20	25	10	35	0.30
▫ Bottom slab (general)	20	20	20	10	30	0.40
▫ Underside of bottom slab	20	20	20	10	35[5.)]	0.40
-	mm	mm	mm	mm	mm	mm

Footnotes:
[1.)] TSFS table 12.1
[2.)] SS-EN 1992-1-1 section 4.4.1.2 table 4.2
[3.)] SS-EN 1992-1-1 section 4.4.1.3
[4.)] TSFS tabele 12.2
[5.)] TSFS chapter 12 paragraph 3§ k_1= c_{min} + 15 mm vhen casting against building foil.

Design parameters high corrosion sensitive reinforcement (pretension):

$c_{min.dur}$: minimum cover with regard to environmental impact

$c_{min.b}$: minimum cover with regard to adhesion requirements
Δc_{dev} : execution tolerance
$c_{min} = \max(c_{min.b}; c_{min.dur}; 10mm)$: SS-EN 1992-1-1 eq. 4.2
$c_{nom} = c_{min} + \Delta c_{dev}$: SS-EN 1992-1-1 eq. 4.1, noted as BM on the drawing

Construction part	$c_{min.dur}$ [1.)]	$c_{min.b}$ [2.)]	c_{min}	c_{dev} [3.)]	c_{nom}	$w_{k,till}$ [4.)]
Superstructure:						
▫ Top bridge deck	25	90	90	10	100	*
▫ Other part of bridge deck	25	90	90	10	100	*
	mm	mm	mm	mm	mm	mm

Footnotes:
[1.)] TSFS table 12.1
[2.)] SS-EN 1992-1-1 section 4.4.1.2 (3) specifies pretension tube ϕ90
[3.)] SS-EN 1992-1-1 section 4.4.1.3
[4.)] TSFS table 12.2 states that crack width is not needed when ”tensile stress” for SLS-F is less than $f_{ctk.0.05}/\zeta$

2. SYSTEM ANALYSIS

2.1 GENERAL

The bridge is built using reinforced concrete.

The bridge is designed as frame bridge with longitudinal beams in superstructure.

Stiffness of bridge deck in longitudinal direction is verified by determining effective width due to shear lag.

Abutments are defined by using shell elements applied to nodal surface in abutments.

Bridge deck is defined by using shell elements applied to nodal surface in superstructure.

Longitudinal beams are defined using beam elements applied to nodal line in superstructure.

Longitudinal beam will be connected to bridge deck using rigid constraints (= Tied Mesh).

FEM-program has the capability of analysing several different static systems within one FEM-model. This bridge use two different analysis (*Analysis 1* & *Analysis 2*). The geometry is the same for both, however the rigid constraints used in superstructure vary.
Analysis 1 is called “Base Analysis”. All changes in *Analysis 1* are based on changes from *Analysis 1*. Evaluation of traffic is performed on *“Base Analysis”*.

For load cases shrinkage (KRYMP) and temperature (TEMP) each longitudinal beam is attached one line of nodes in superstructure. This to avoid large internal forces in deck due to prevented contraction in transversal direction. These loads are performed in *Analysis 2.*

For all other load cases each longitudinal beam is attached two lines of nodes in superstructure. The reason for this is to get correct load effect in transversal direction, since with only one line the span length for deck would be too long. These loads are performed on *Analysis 1.*

Entire structure is modelled using isotropic material.

Bridge foundation consists of compacted gravel.

Wingwalls are not modelled statically since considered inactive in vertical direction. This is due cracking and the use of only minimal reinforcement in this direction.

Edge beams are not modelled statically since considered inactive. This assumption is considered on safe side. The assumption will facilitate future replacement of edge beams.

Bottom of abutments will be connected to fictive rigid beams using rigid constrains (= Tied Mesh). The rigid beams will have infinite stiffness in all directions apart from axial. In this direction stiffness will be negligible.

The end of longitudinal beams in superstructure are only to substructure to one node at location where each nodal line meets the shell elements in the abutments. To spread forces to substructure as accurately as possible fictious rigid beams are introduced at these connection points. The rigid beams (=Rigid beam superstructure) will have infinite stiffness in all directions apart but axial. In this direction stiffness will be negligible.

At bottom of abutments a fictitious rigid beam (= Rigid beam abutment) with infinite stiffness in all directions apart from axial. In this direction stiffness will be negligible. In the middle of this fictious rigid beam a super node will be connected. This to retrieve reactions a single point for every support.

Shell elements are used to define Superstructure end zone. To be able to apply prestress tendons in this area fictitious beams (= Weak beam superstructure) are applied with negligible stiffness in all directions.

In system analysis gross cross section may be used for longitudinal beams in superstructure, see SS-EN 1992-1-1 section 5.3.2.1 point (4).

In FEM-analysis constant cross section is used for longitudinal beams along entire span length, see SS-EN 1992-1-1 section 5.3.2.1 (4).

The difference in stiffness between conventionally reinforced construction elements shall be considered for statically indeterminate prestressed concrete structures. According to older technical practice, the stiffness for conventionally reinforced construction elements is 60% of that applicable to prestressed construction elements. This interpretation aligns with what is stated in TRVINFRA-003331 section 10.1.2.2. This technical practice is applied to the studied bridge.

The difference in stiffnesses is handled by using 60 % of E-modulus for shells in substructure.

For the deck in superstructure the 60 % of E-modulus in transversal direction is used. In the longitudinal direction 100 % of E-modulus is considered. This requirement requires use of orthotropic material.

For longitudinal beams in superstructure 100 % of E-modulus is considered.

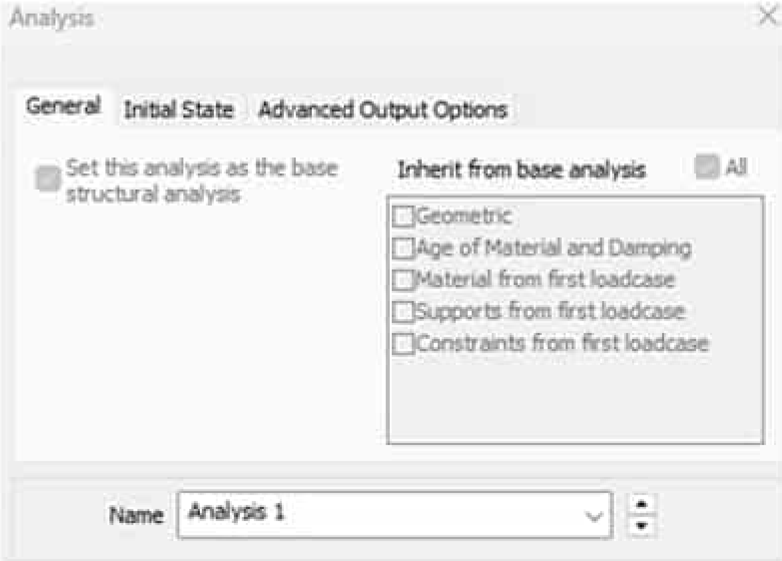
Attachments:

Attachment	Name
1	Input receipt
2	Results reactions
3	Results abutments
4	Results bridge deck
5	Results longitudinal beams

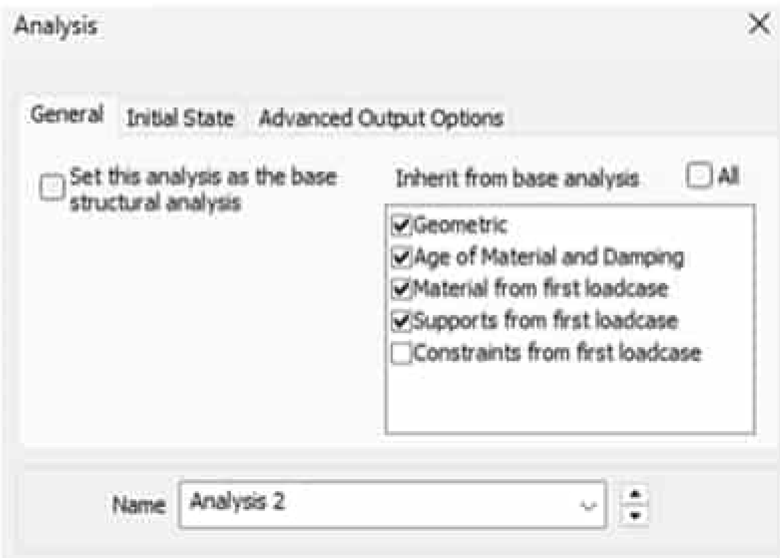
Table analysis:

Nr	Name	Remark
1	Analysis 1	Base Analysis
2	Analysis 2	Shrinkage & Temperature

Definition analysis 1:



Definition analysis 2:



2.2 SKETCH SYSTEM ANALYSIS

2.2.1 Geometry

In order to describe geometry first POINTS are defined.

Beam elements are defined by applying attributes to LINES.

Shell elements are defined by applying attributes to SURFACES.

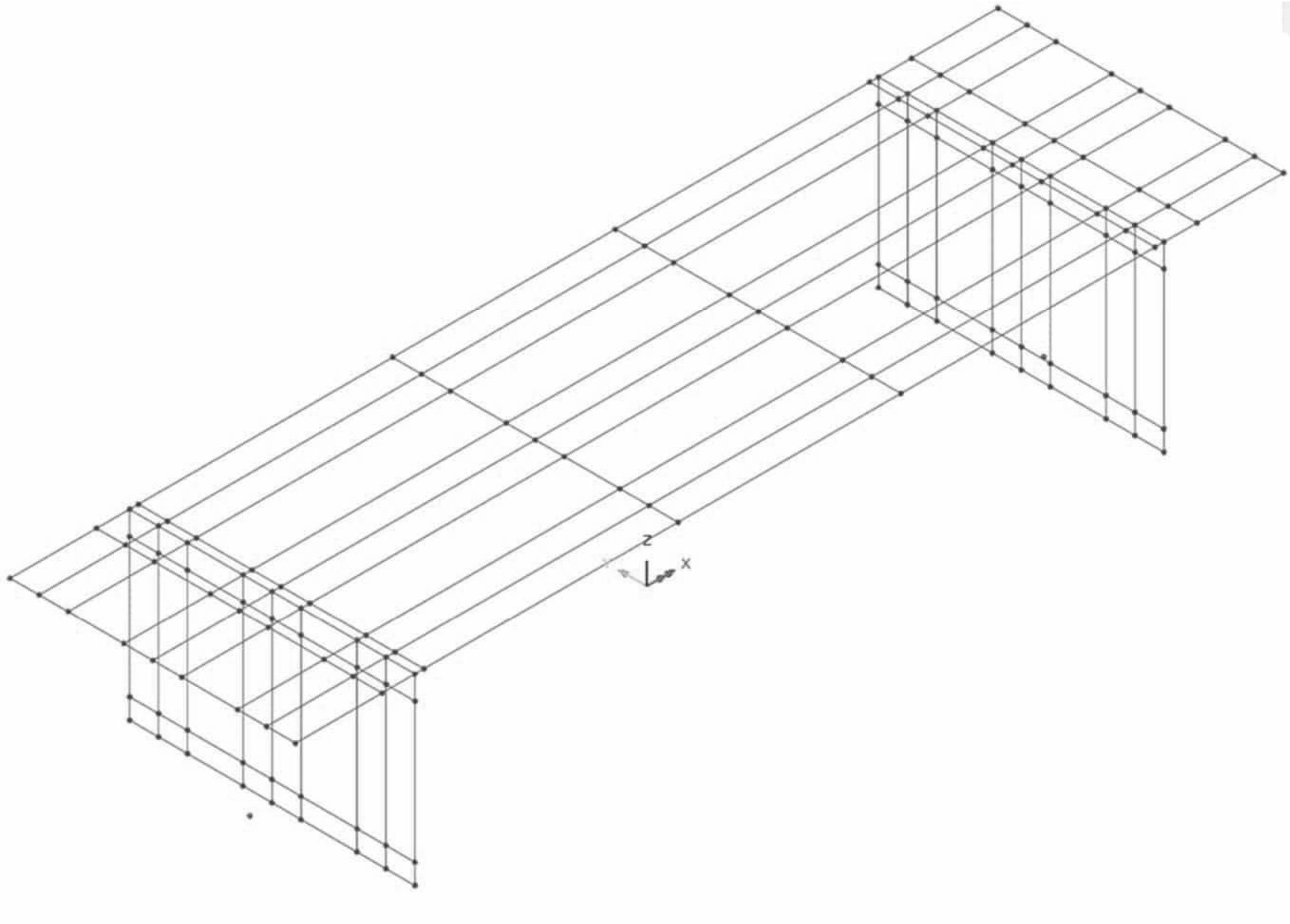
Attached pictures are retrieved from graphical sketches generated by FEM-program of POINTS, LINES and SURFACES.

All coordinates needed to describe POINTS are found in attachment 1.

All POINTS needed to describe LINES are found in attachment 1.

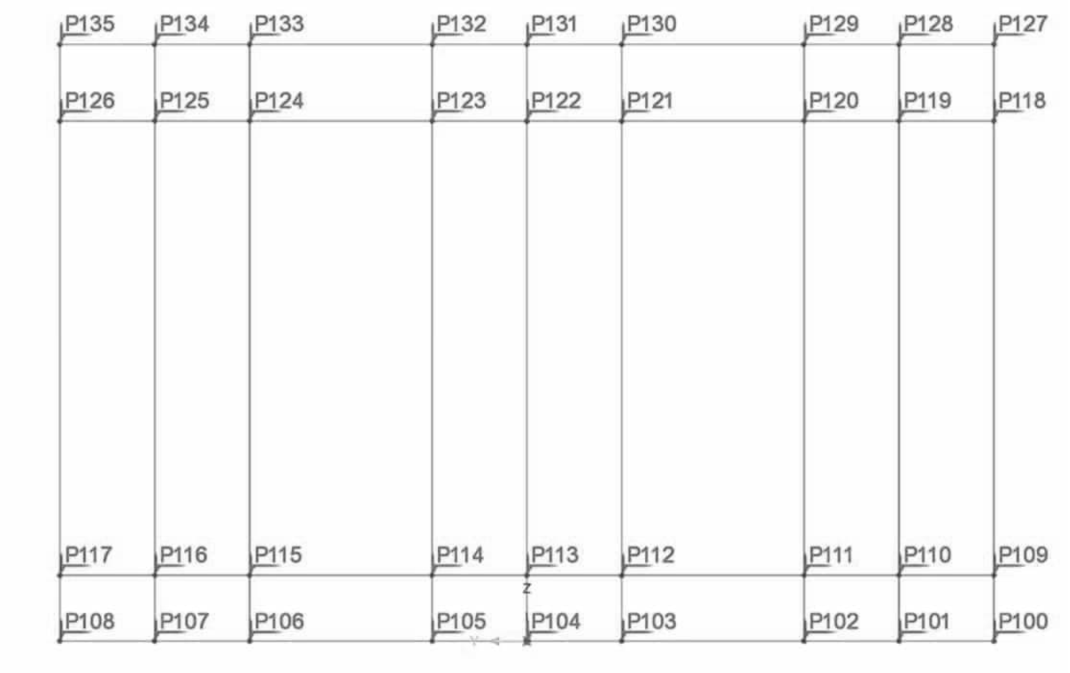
All LINES need to describe SURFACE are found in attachment 1.

Overview :

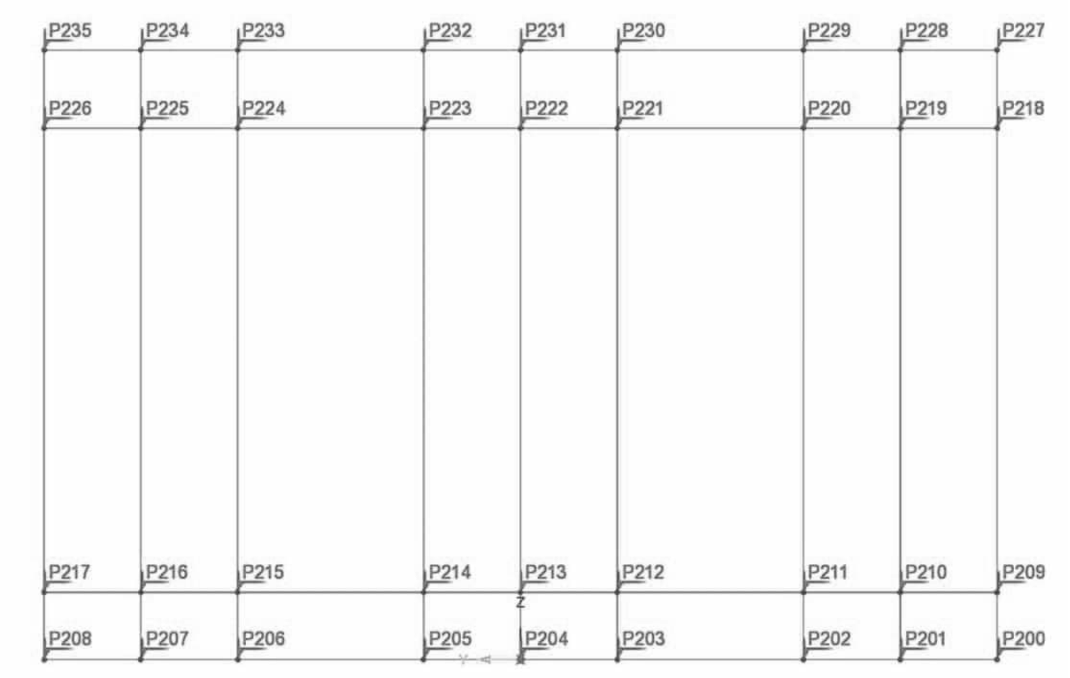


2.2.1.1 Geometry : POINTS

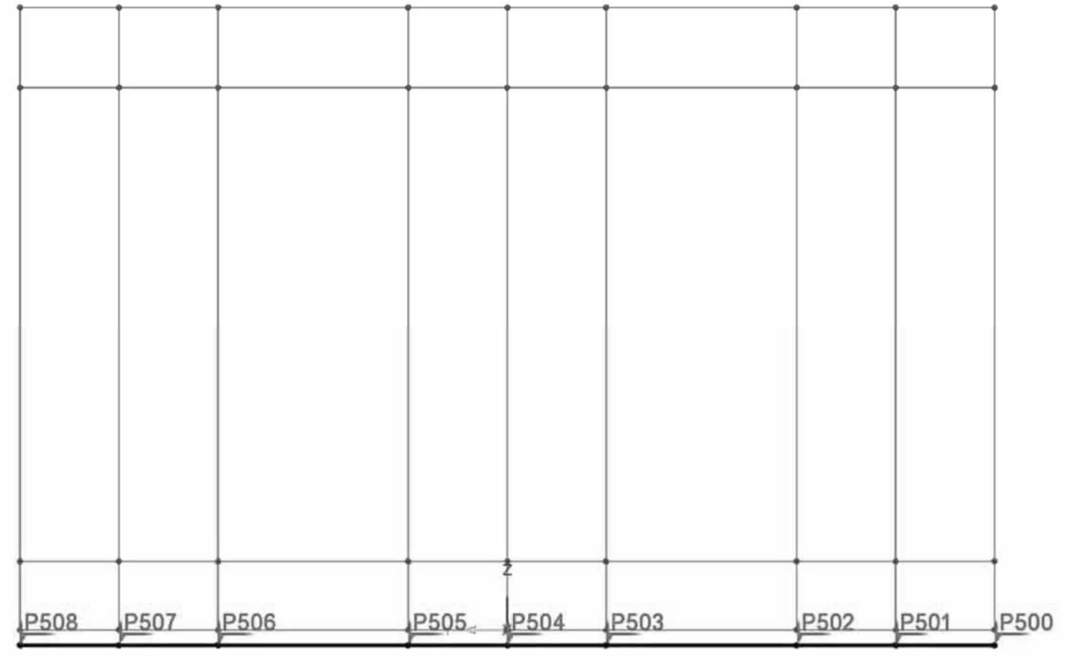
Abutment 1 :



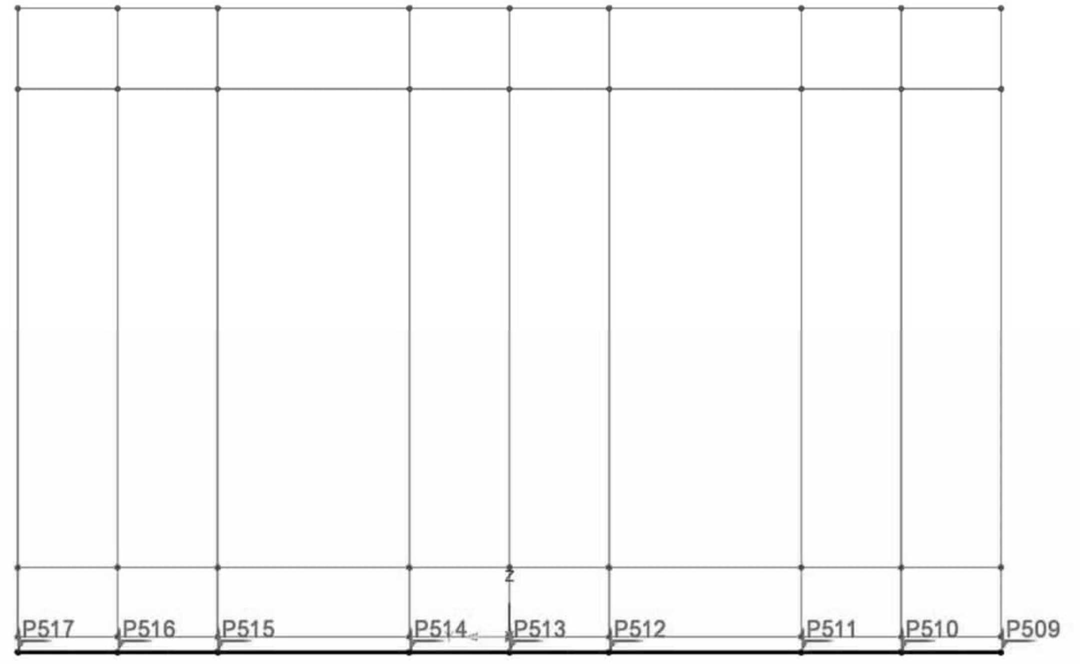
Abutment 2 :



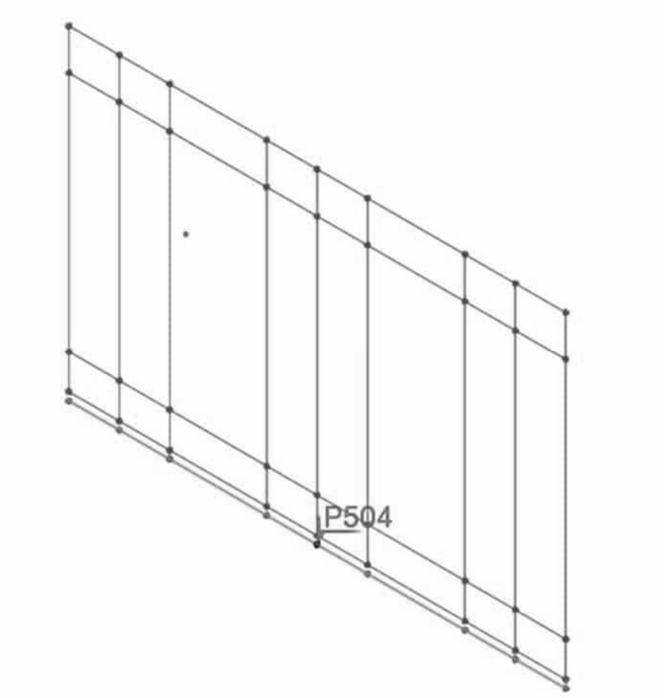
Rigid beam at abutment 1 :



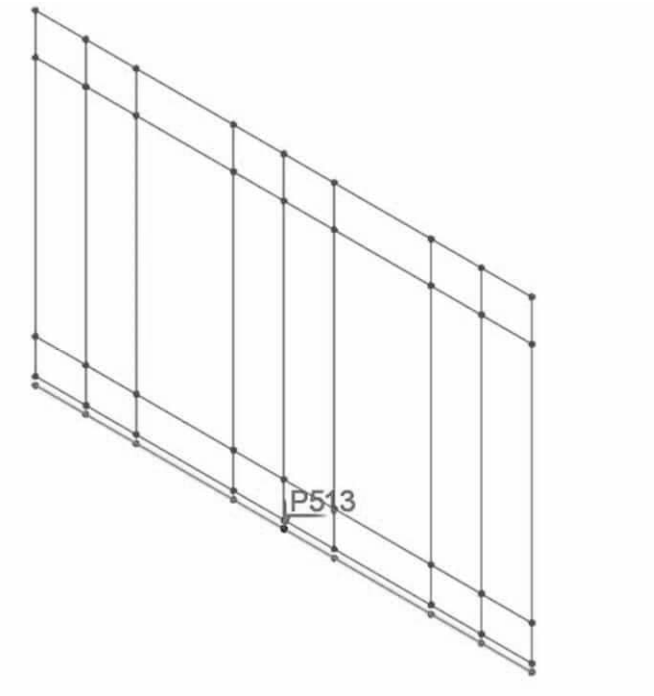
Rigid beam at abutment 2 :



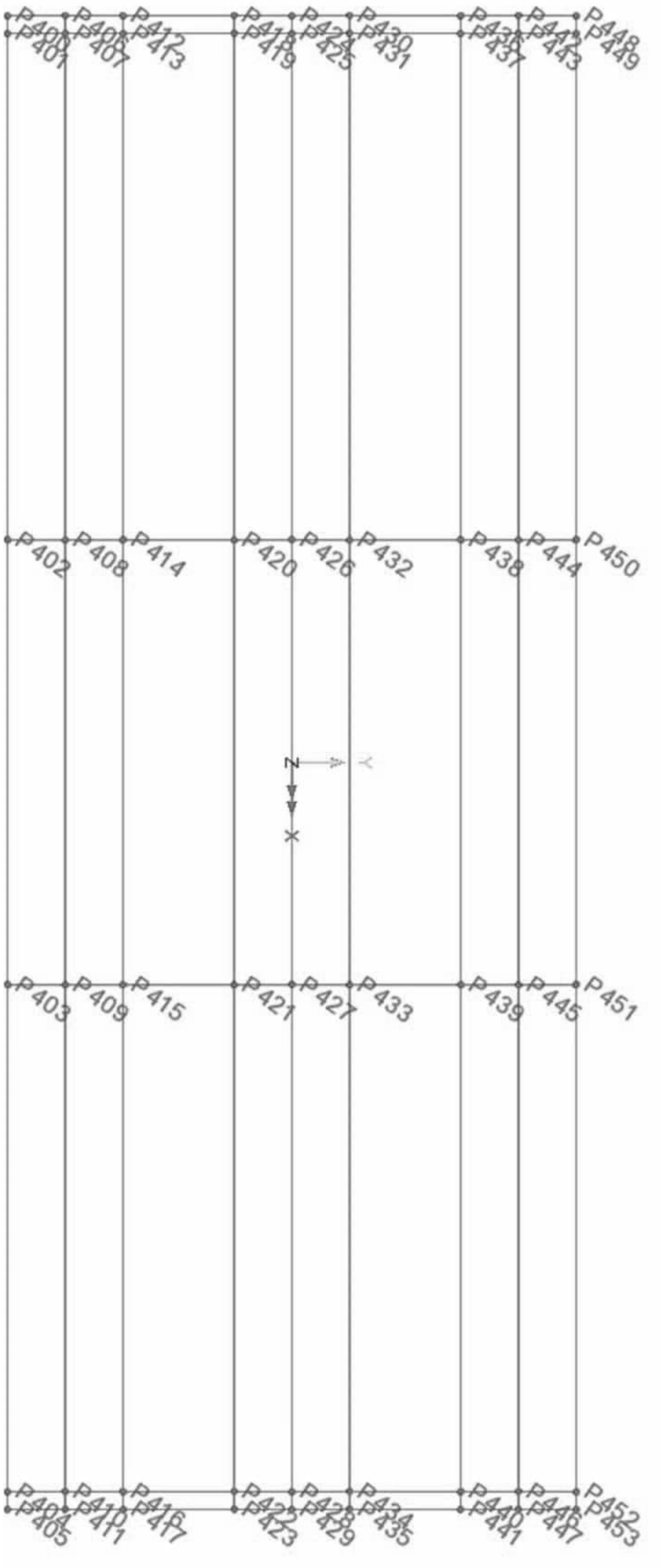
Supernode - point foundation 1 :



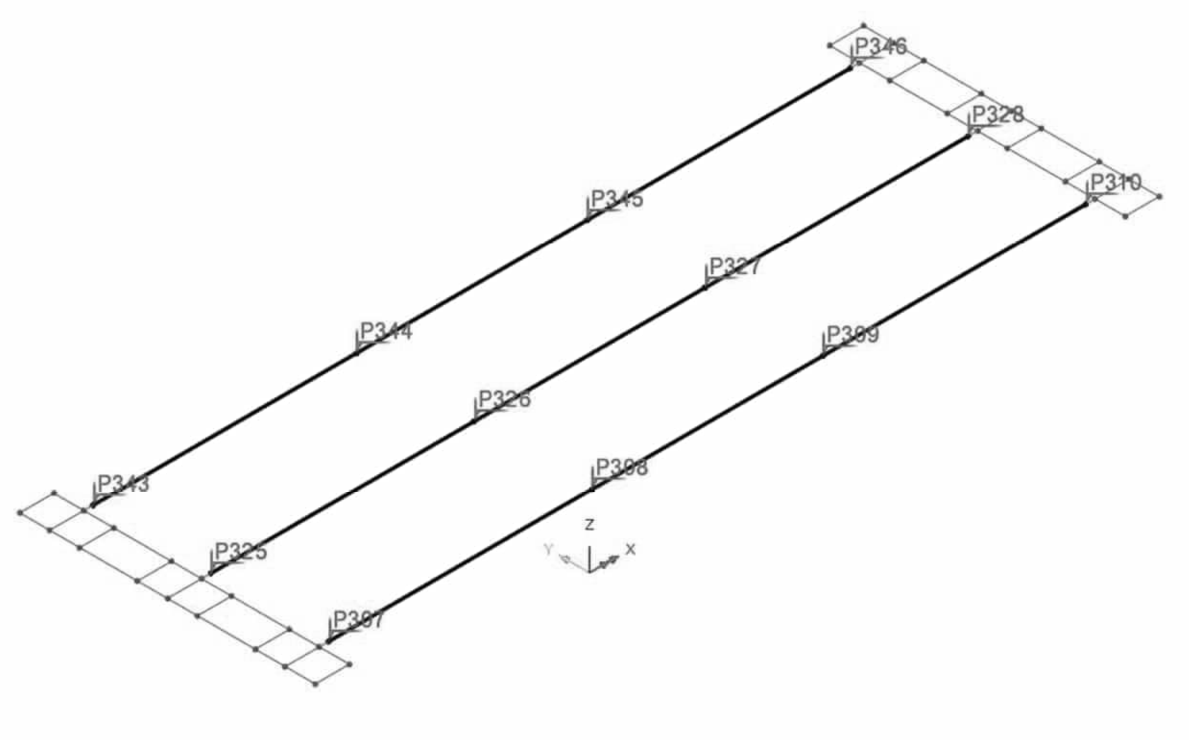
Supernode - point foundation 2 :



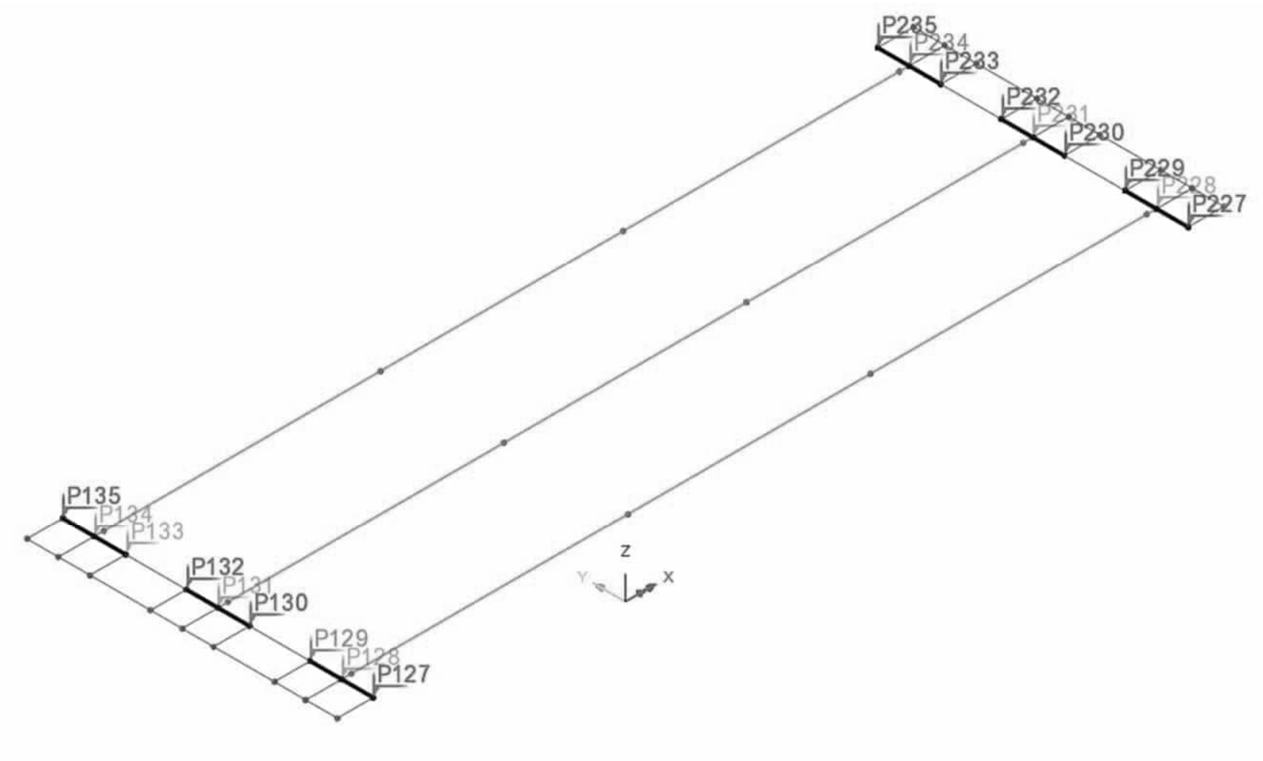
Superstructure - deck :



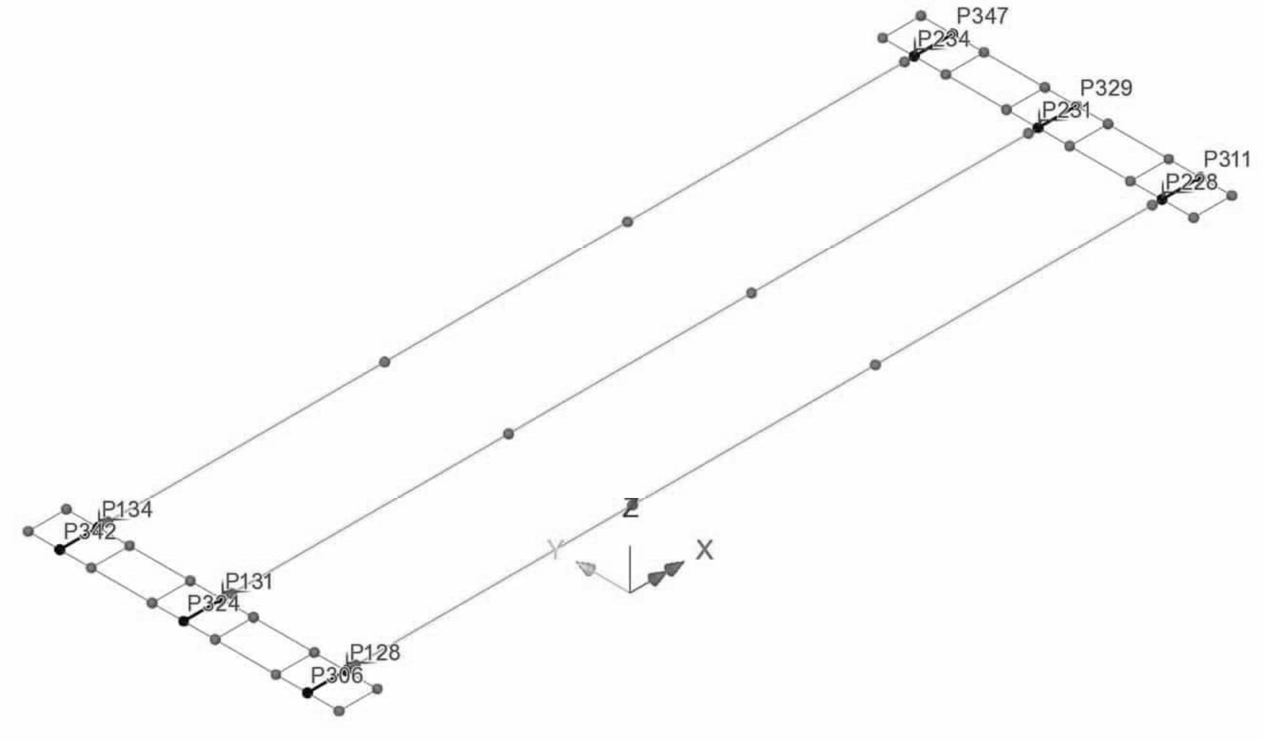
Superstructure - longitudinal beams :



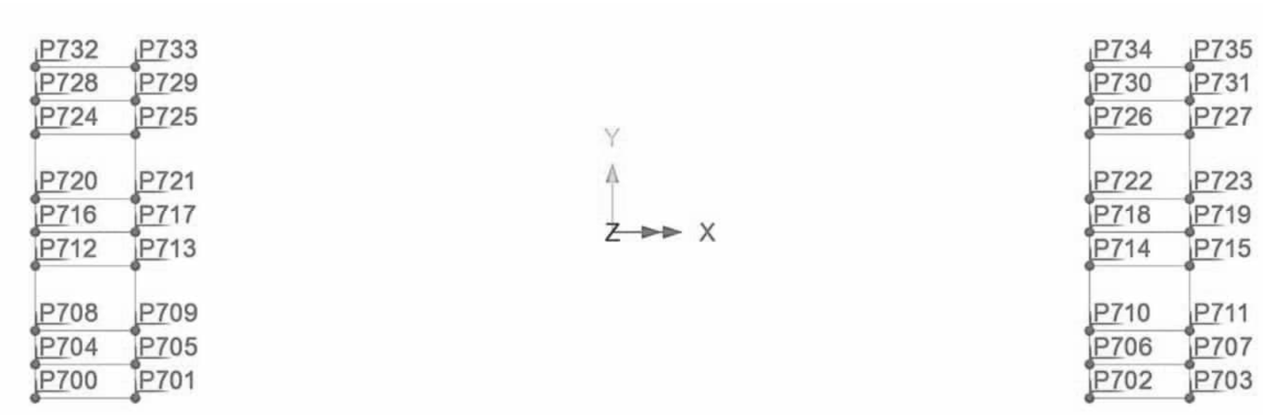
Superstructure – rigid beams :



Superstructure – weak beams:



Superstructure – link slab :



2.2.1.2 Geometry : LINES

See input receipt (attachment 1).

2.2.1.3 Geometri : SURFACES

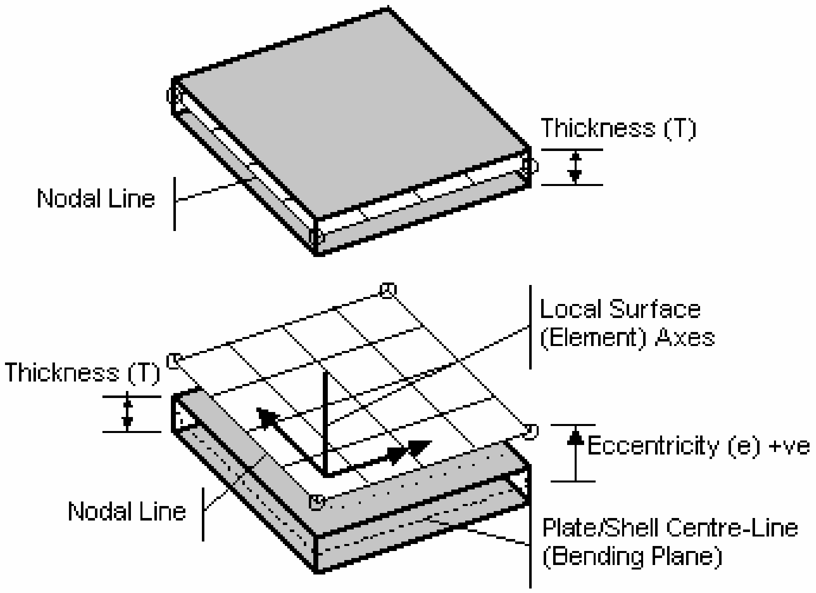
See input receipt (attachment 1).

2.3 CROSS SECTION PROPERTIES

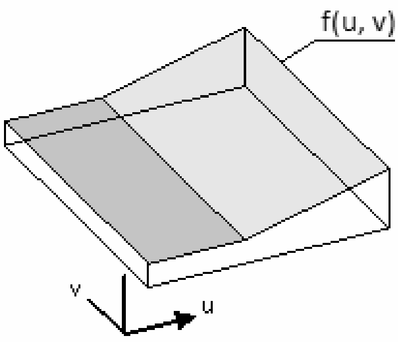
By experience stiffness increases by 1:3 at all joints as seen below.

2.3.1 Shell element

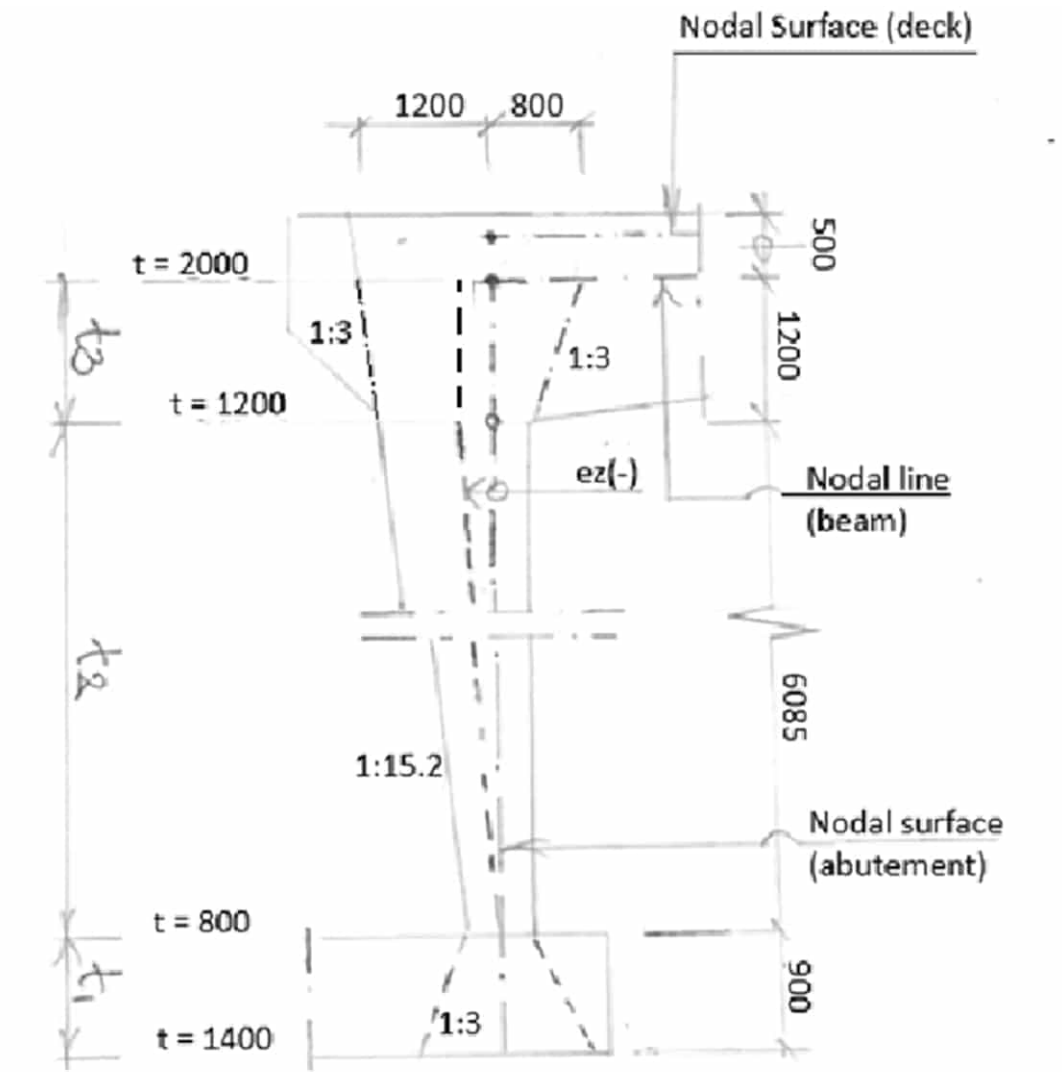
Principle figures of geometry associated to shell elements (”Thick shell” / QTS4) are seen below.



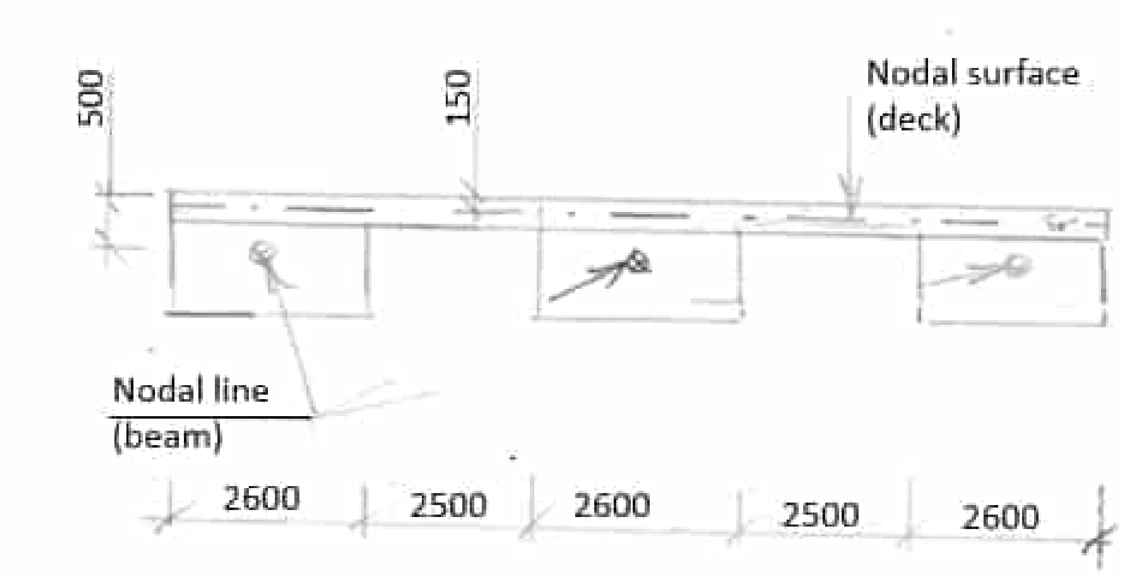
Varying thickness in shell element is handled using ”Function variation”. This makes it possible to create a function f(u,v) as seen below.

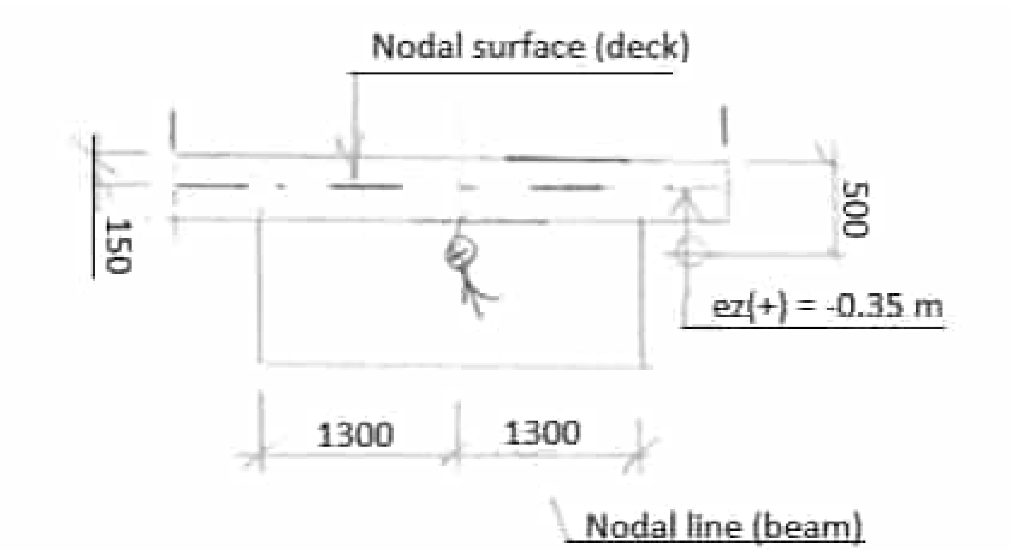


Geometry abutments:

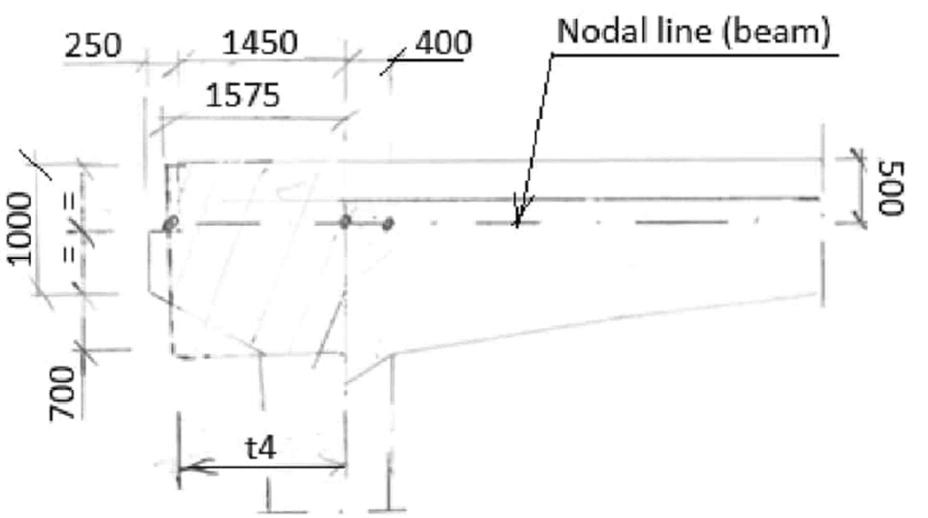


Geometry superstructure – deck :





Geometry superstructure end zone:



Surface function thickness :

Variation	function(u,v)	Anm.
t1	1.40-0.60·u	Slab
t2	0.80+0.40·u	Abutment
t3	1.20+0.80·u	Abutment – Joint areas
-	m	-

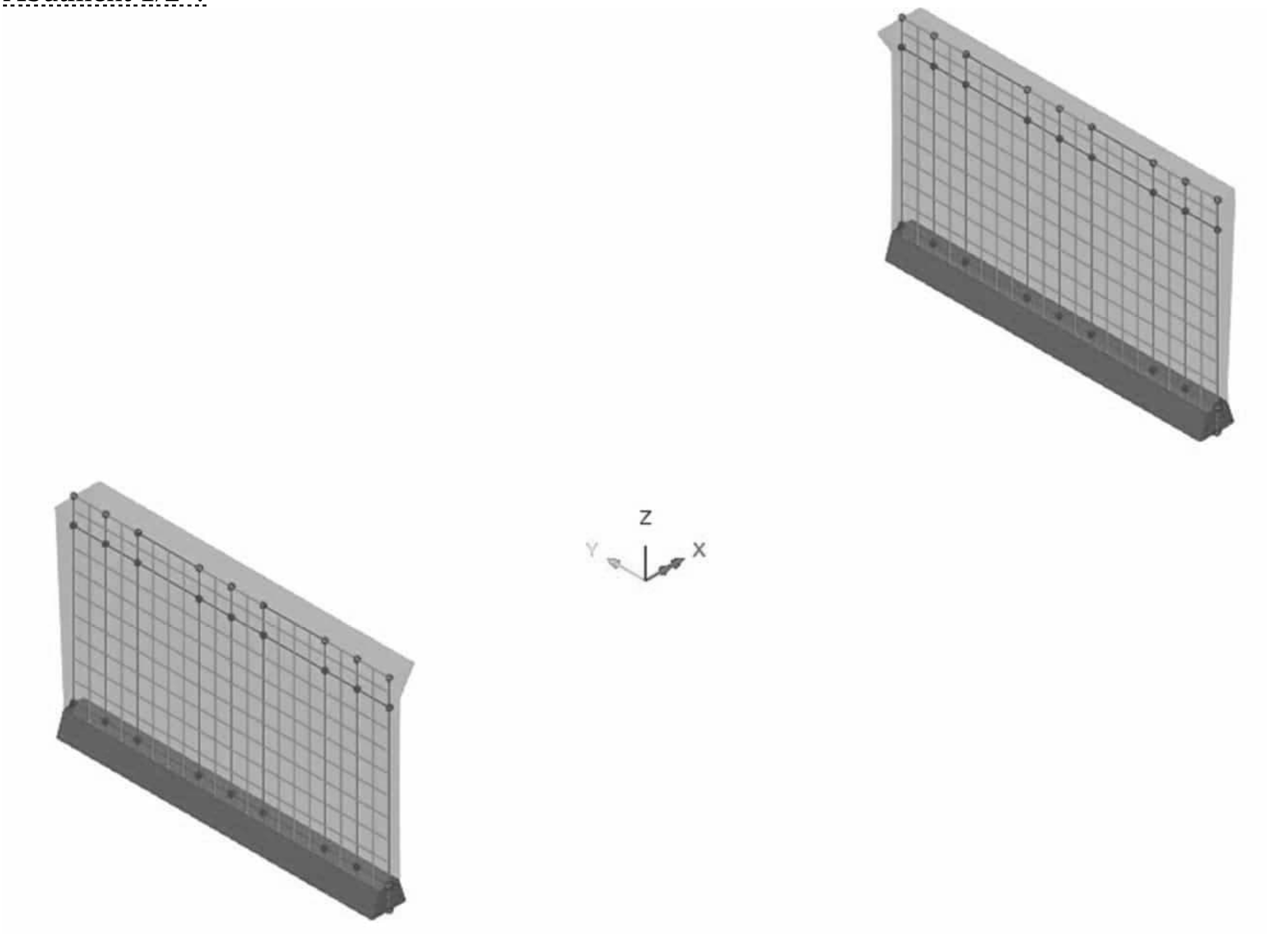
Surface function eccentricities:

Variation	Function(u,v)	Anm.
e2	-0.20·u	Abutment 1
-e2	0.20·u	Abutment 2
e3	-0.20	Abutment 1 – Joint area
-e3	0.20	Abutment 2 – Joint area
-	m	-

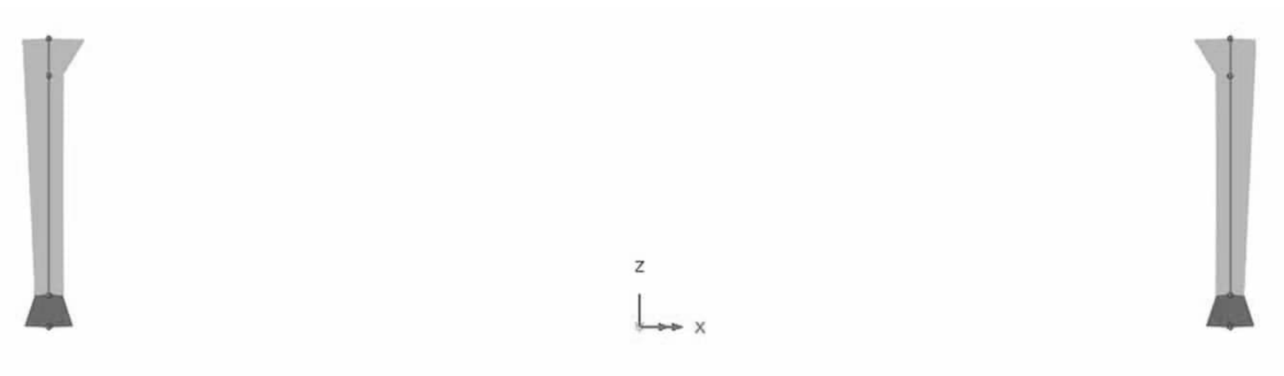
Surface geometry :

Attribute	t	ez	Anm.
t = 0.30 m	0.30	0	Superstructure deck
t1	t1	0	Slab
t21	t2	e2	Abutment 1
t31	t3	e3	Abutment 1 – Joint area
t22	t2	-e2	Abutment 2
t32	t3	-e3	Abutment 2 – Joint area
t4	1.70	0.350	Superstructure end zone
-	m	m	-

Abutment 1/2 :

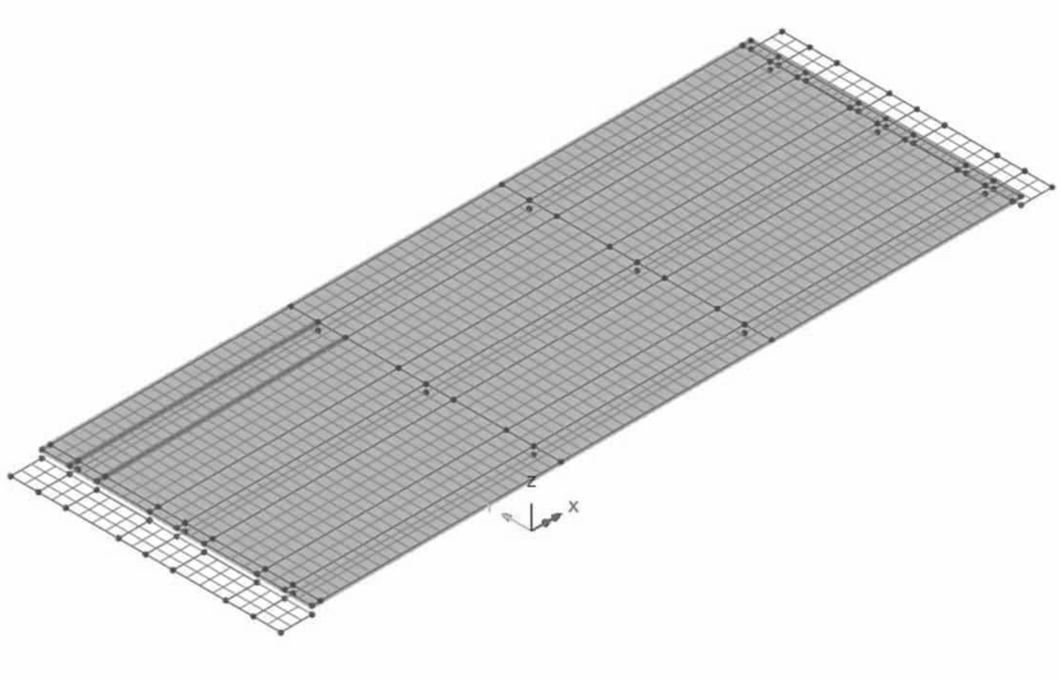


Overview

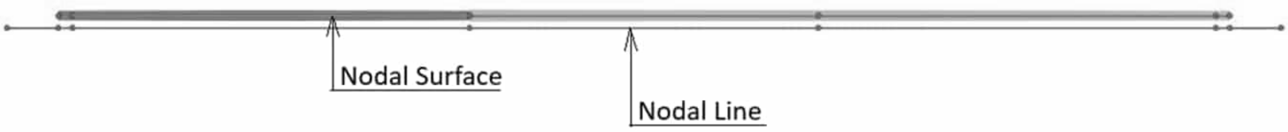


Elevation

Deck - superstructure:

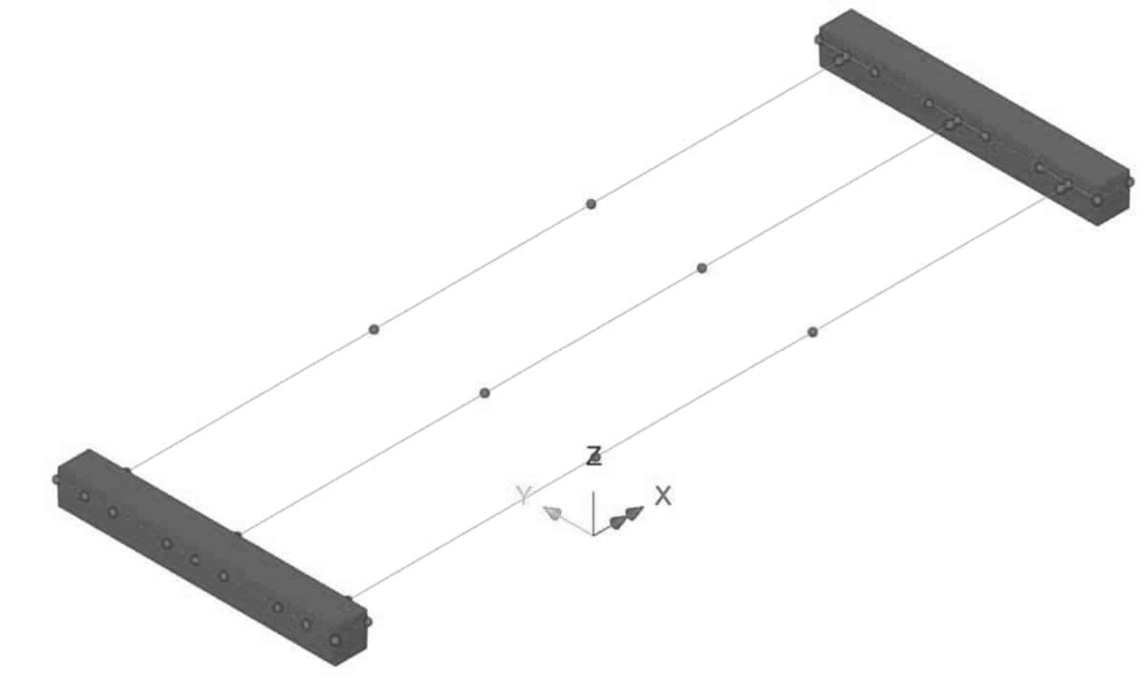


Overview

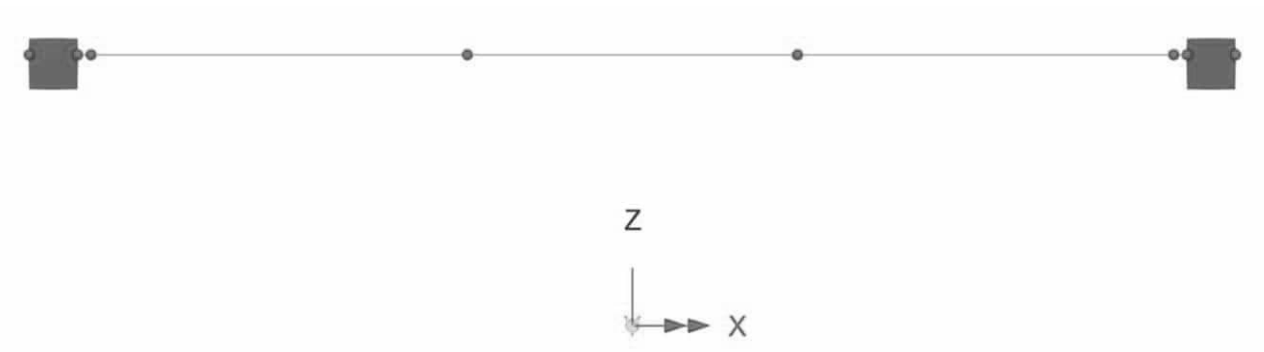


Elevation

Superstructure end zone:



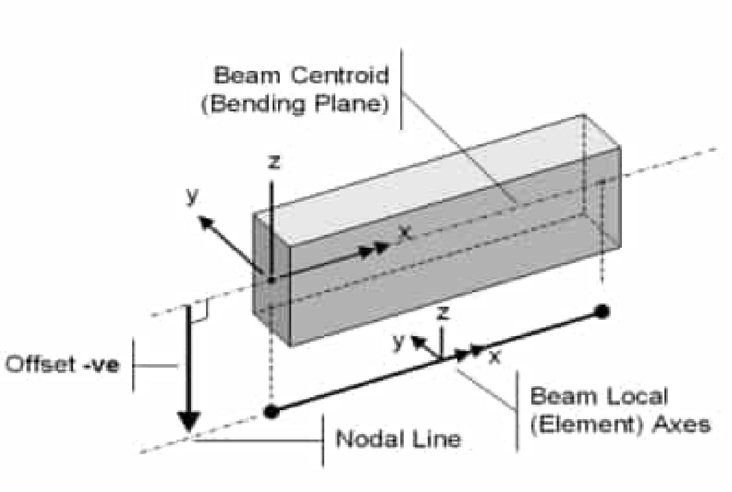
Overview



Elevation

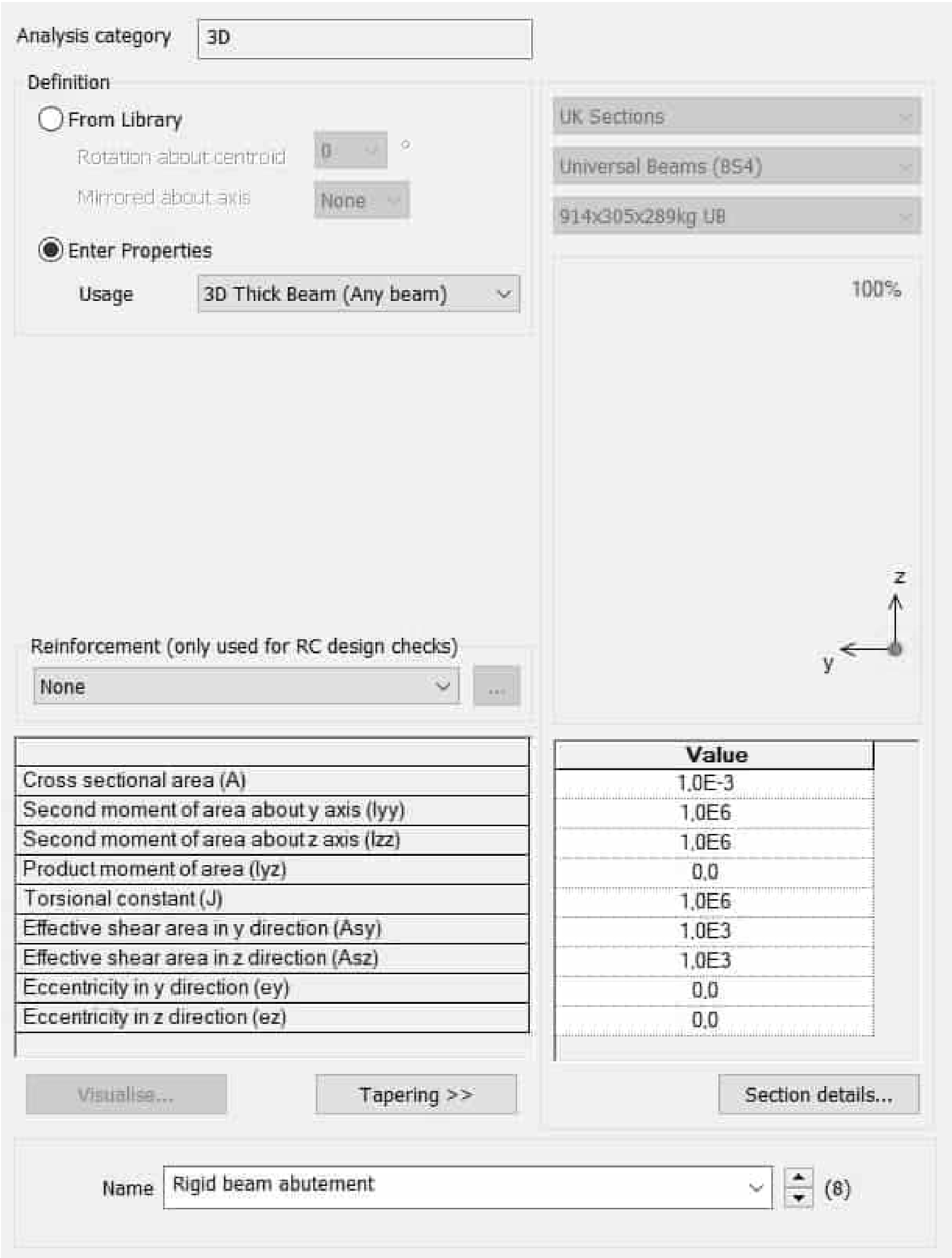
2.3.2 3D-beams (”Thick beam” / BMS3)

Principal sketch of geometry associated to 3D beam elements are seen below.



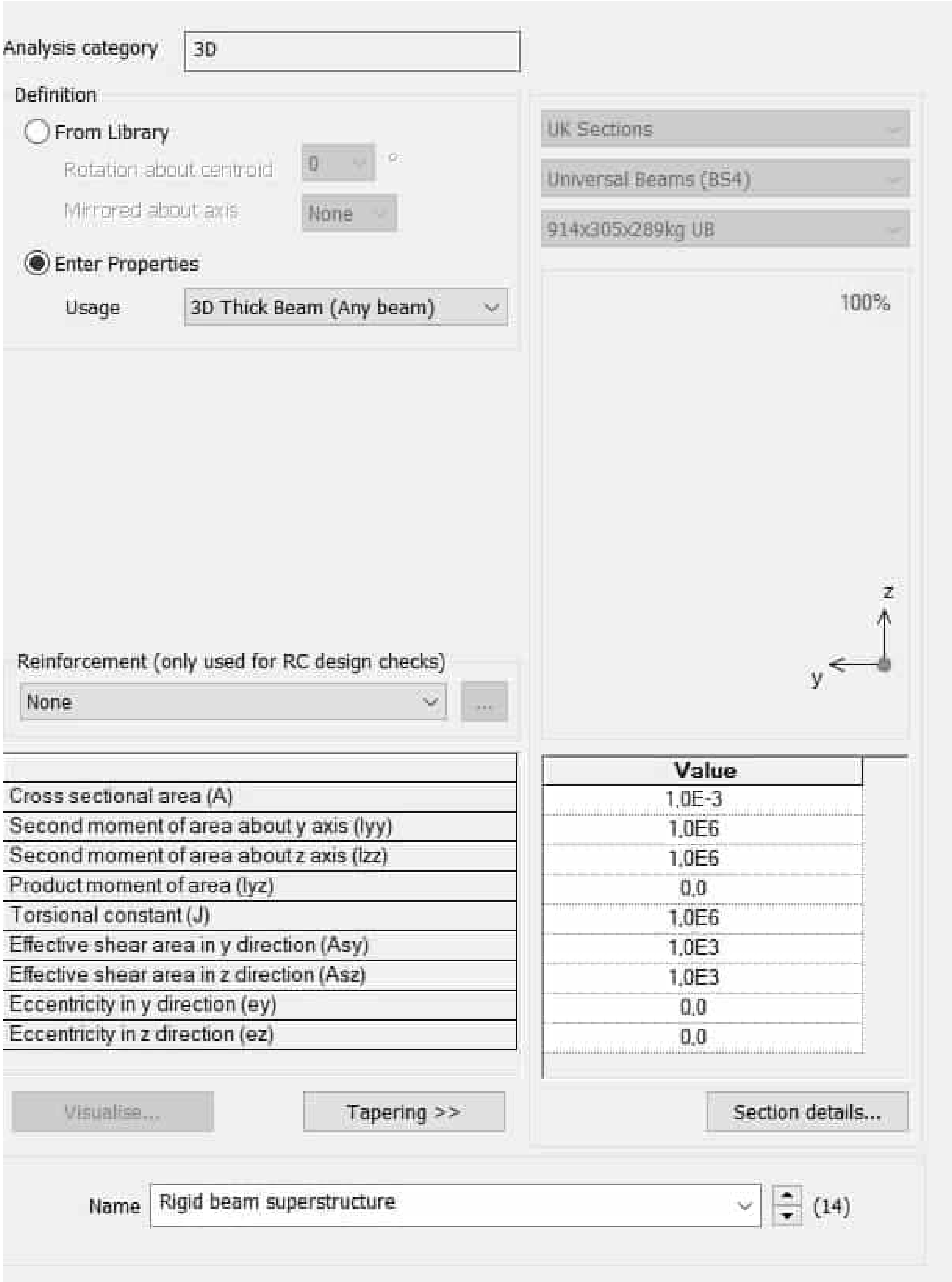
2.3.2.1 Rigid beam abutment

A fictive rigid beam is introduced at bottom of each abutment. The beam has infinite stiffness in all direction apart from axial direction. In this direction stiffness is negligible.



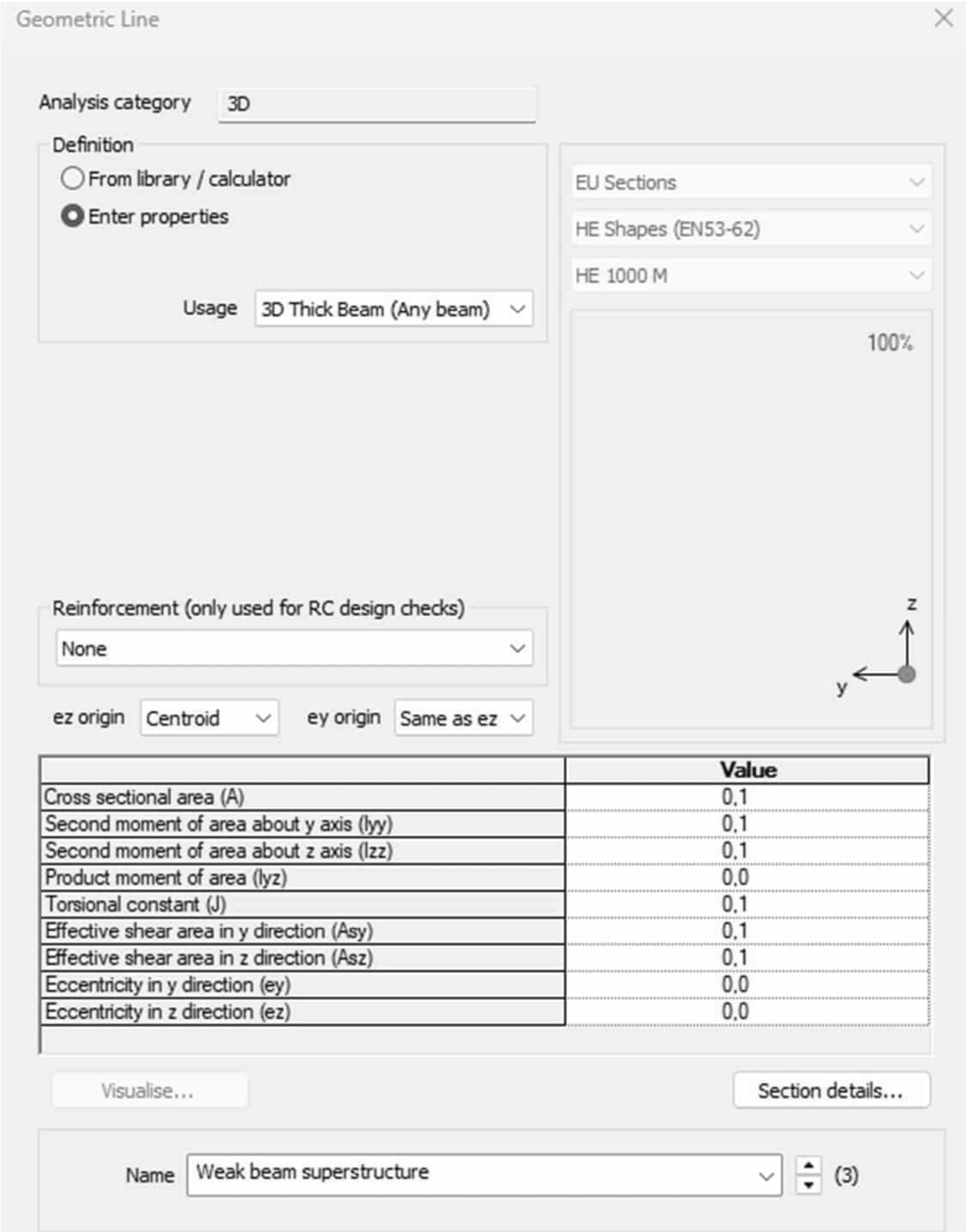
2.3.2.2 Rigid beam - superstructure

Rigid beams are introduced at top of abutment at location where nodal surface of superstructure is attached. bottom of abutment. The beam has infinite stiffness in all direction apart from axial direction. In this direction stiffness is negligible.



2.3.2.3 Weak beam - superstructure

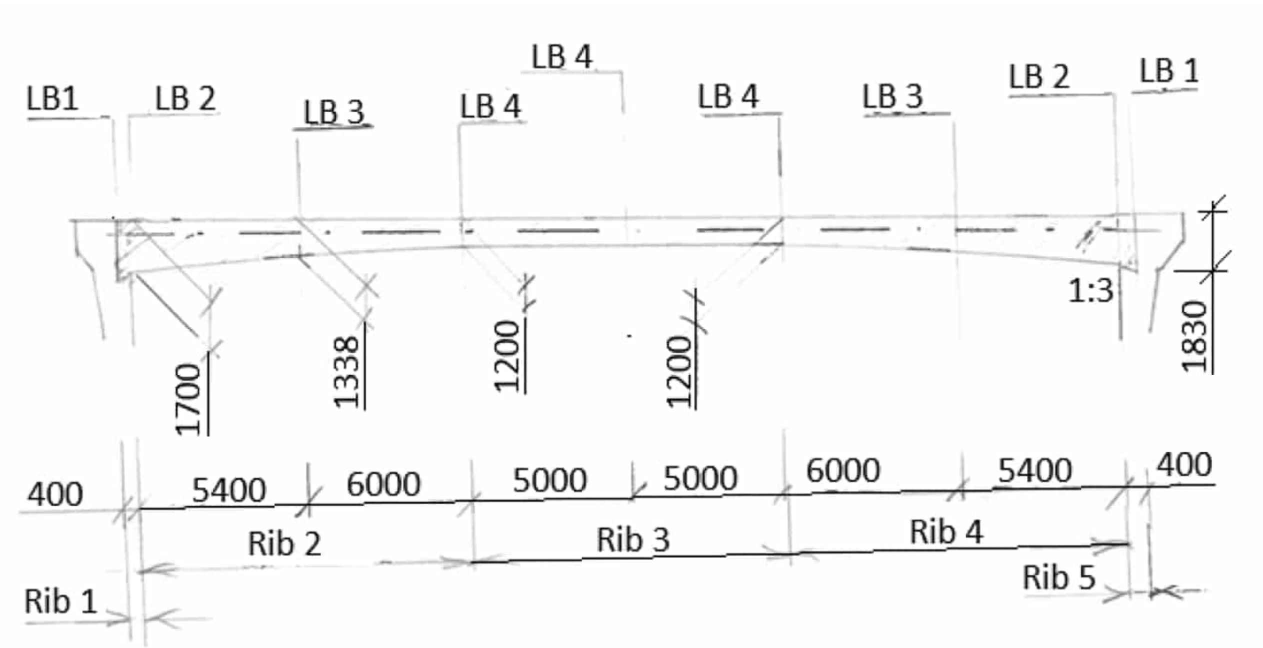
This fictious beam is only used to be able to apply tendons in end zone of superstructure. The beam has negligible stiffness in all directions.



2.3.2.4 Longitudinal beams

The 3 longitudinal beams are divided into 8 minor beams (Rib 1 – Rib 8) with varying height.

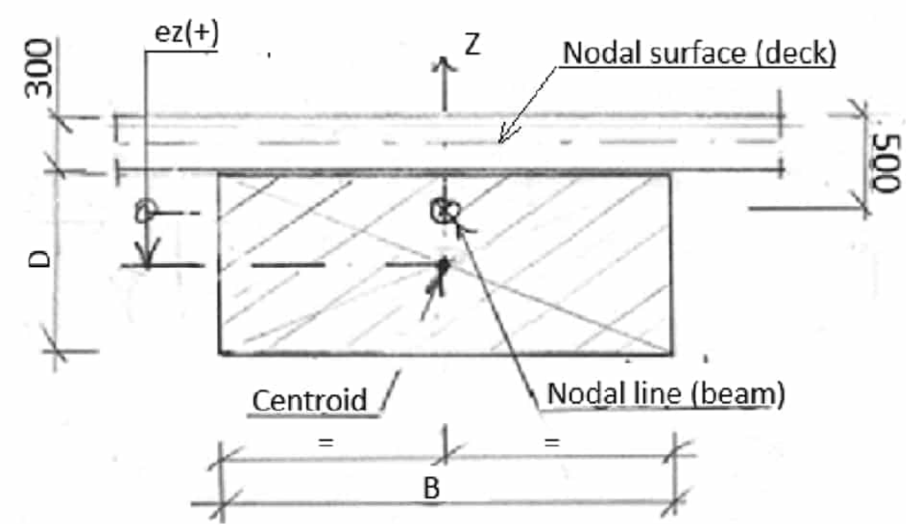
The beams are defined using 9 cross sections (LB1-LB9) as seen below.



Input rectangular cross section (LB1-LB4)

Data	LB 1	LB 2	LB 3	LB 4
H	1.830	1.700	1.338	1.200
D	1.530	1.400	1.038	0.900
B	2.600	2.600	2.600	2.600
-	m	m	m	m

Results rectangular cross section



Data	LB 1	LB 2	LB 3	LB 4	Unit
A	3.98	3.64	2.70	2.34	m^2
Iyy	0.776	0.594	0.242	0.158	m^4
Izz	2.241	2.051	1.520	1.318	m^4
Iyz	0	0	0	0	m^4
J	1.964	1.576	0.726	0.494	m^4
-	m	m	m	m	-

Multiple varying section

e_y : excentricity in y-direction

$e_z = \frac{D}{2} - 0.20m$: excentricity in z-direction

Balk	Section	Distance	ey	ez	Shape interpolation
Rib 1	LB 1	0	0	0.565	Start
	LB 2	0.40	0	0.500	Linear
Rib 2	LB 2	0	0	0.500	Start
	LB 3	5.40	0	0.319	Quadratic
	LB 4	11.40	0	0.250	Quadratic
Rib 3	LB 4	0	0	0.250	Start
	LB 4	10.00	0	0.250	Linear
Rib 4	LB 4	0	0	0.250	Start
	LB 3	6.00	0	0.319	Quadratic
	LB 2	11.40	0	0.500	Quadratic
Rib 3	LB 2	0	0	0.500	Start
	LB 1	0.40	0	0.565	Linear
-	-	m	m	m	-

2.4 MATERIAL

Material properties seen below are to be used.

Substructure C35/45 : E_{cm} = 34 GPa

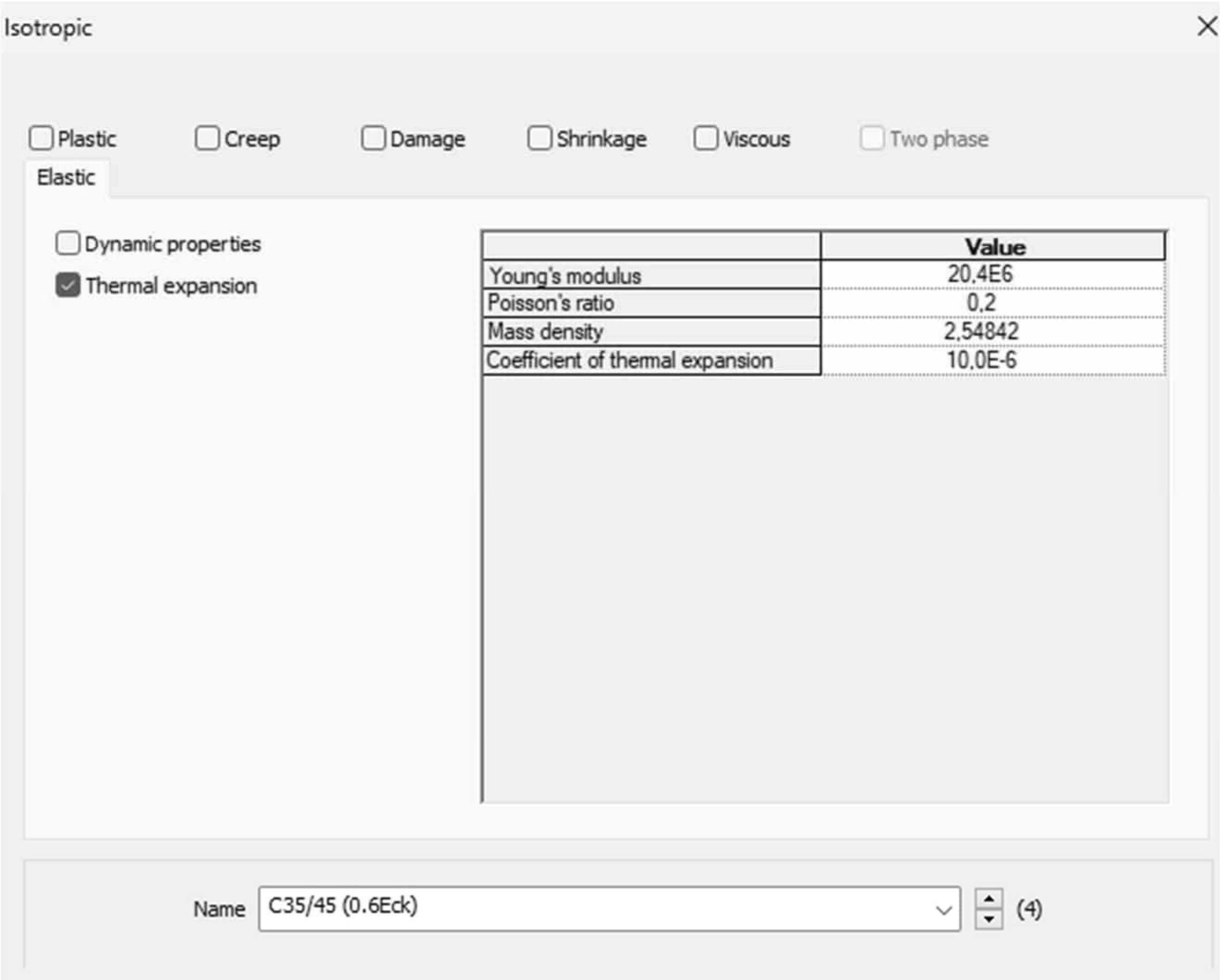
Superstructure C40/50 : E_{cm} = 35 GPa

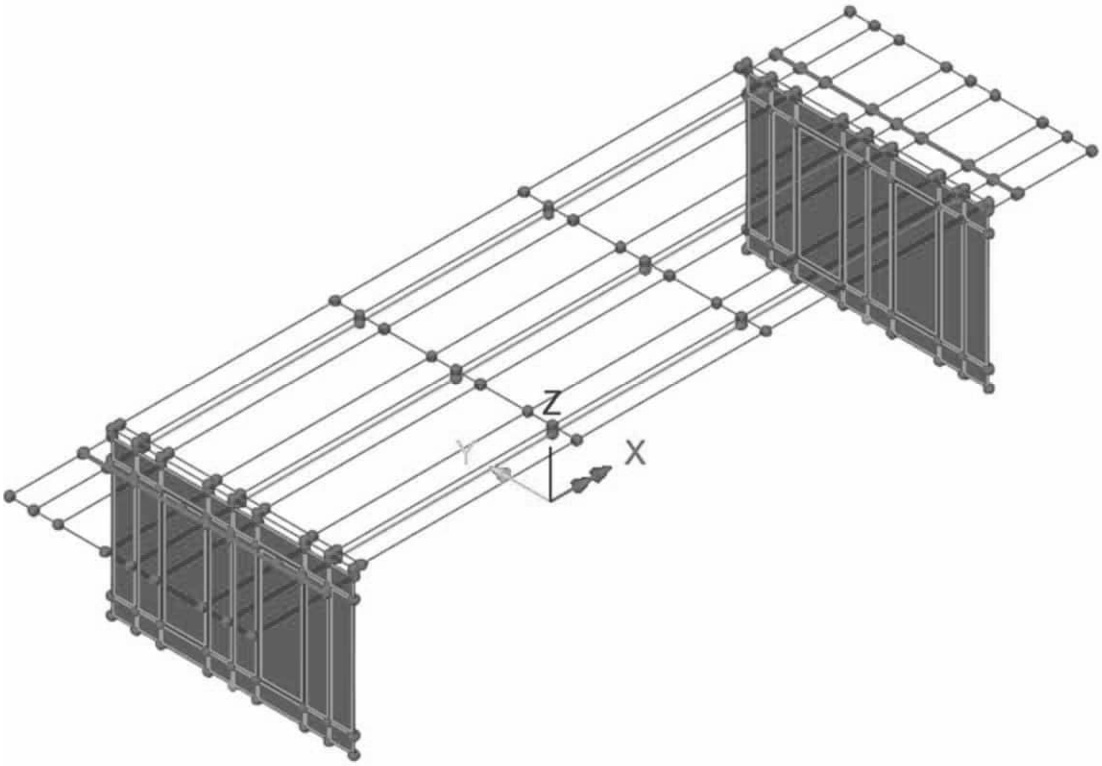
The use of function “Slice Resultant Beams/Shells” does not permit the use of orthropic material. All nodes in nodal surface and nodal line must be isotropic.

2.4.2.1 Material : Isotropic concrete C35/45 reduced (0.6E_{ck})

Applied to elements in substructure. The stiffness is reduced to 40 %.

$E = 0.6 \cdot E_{ck} = 0.6 \cdot 34 \cdot 10^3$ MPa $= 20.4 \cdot 10^3$ MPa



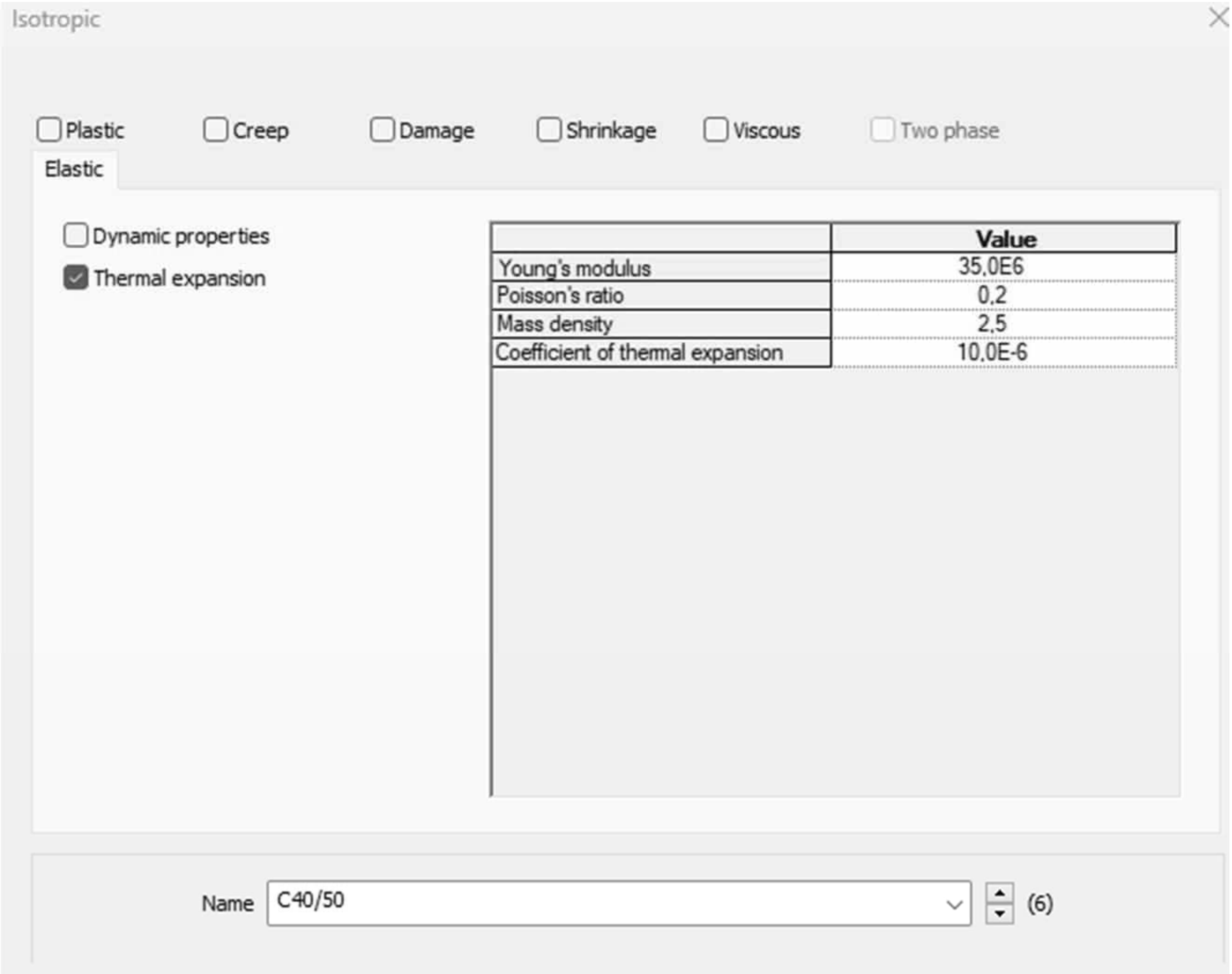


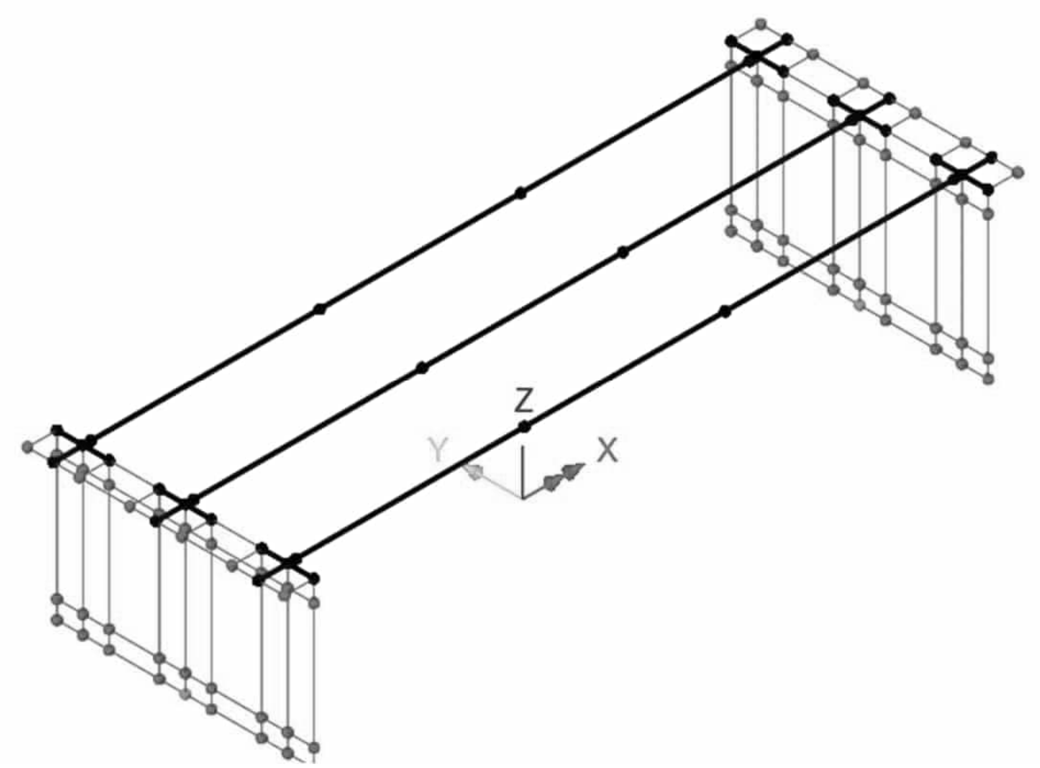
Overview 3D
Applied to abutments and rigid beams.

2.4.2.2 Material : Isotropic concrete C40/50

Applied to longitudinal beams and deck above beam.

$E = 35 \cdot 10^3$ MPa

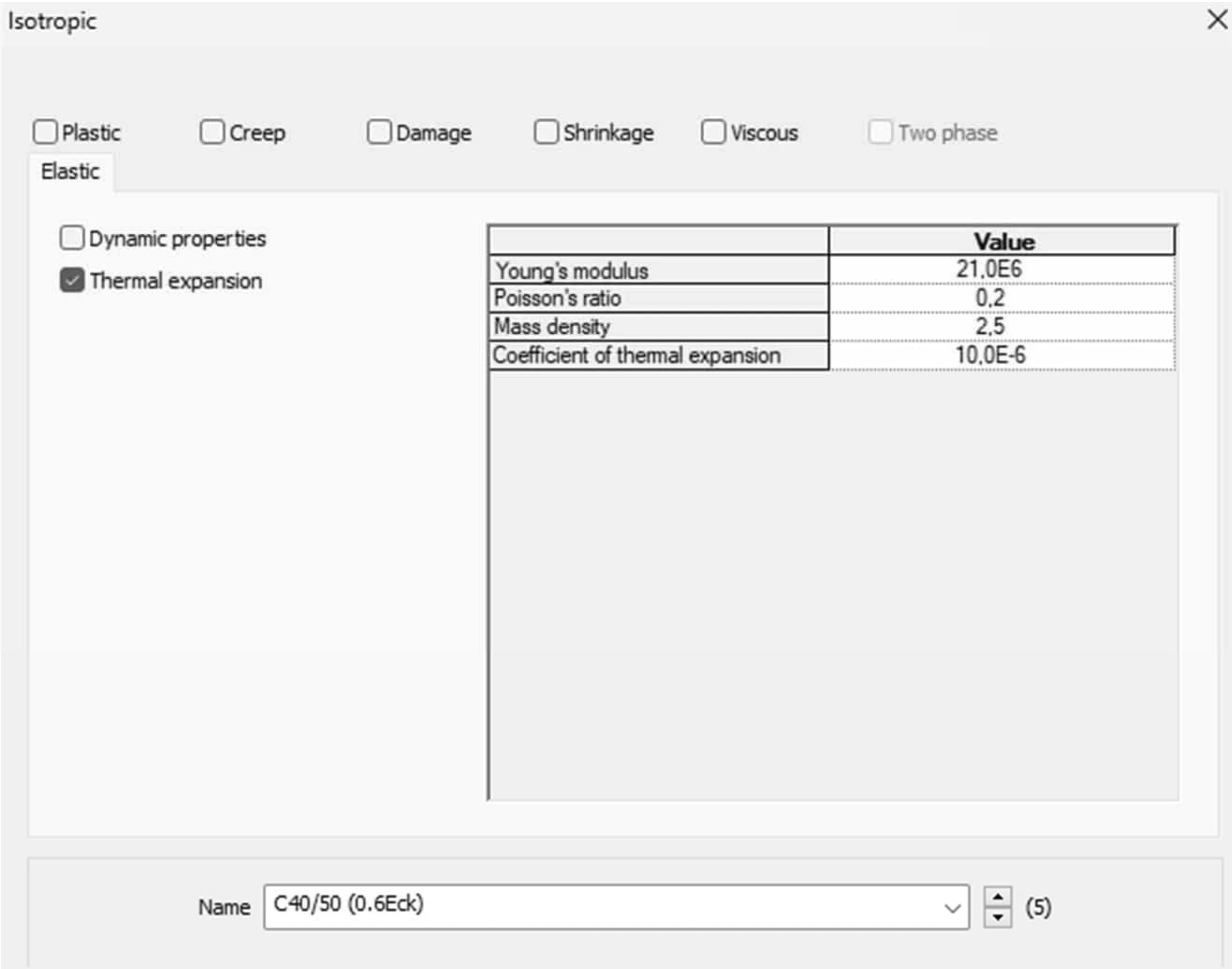


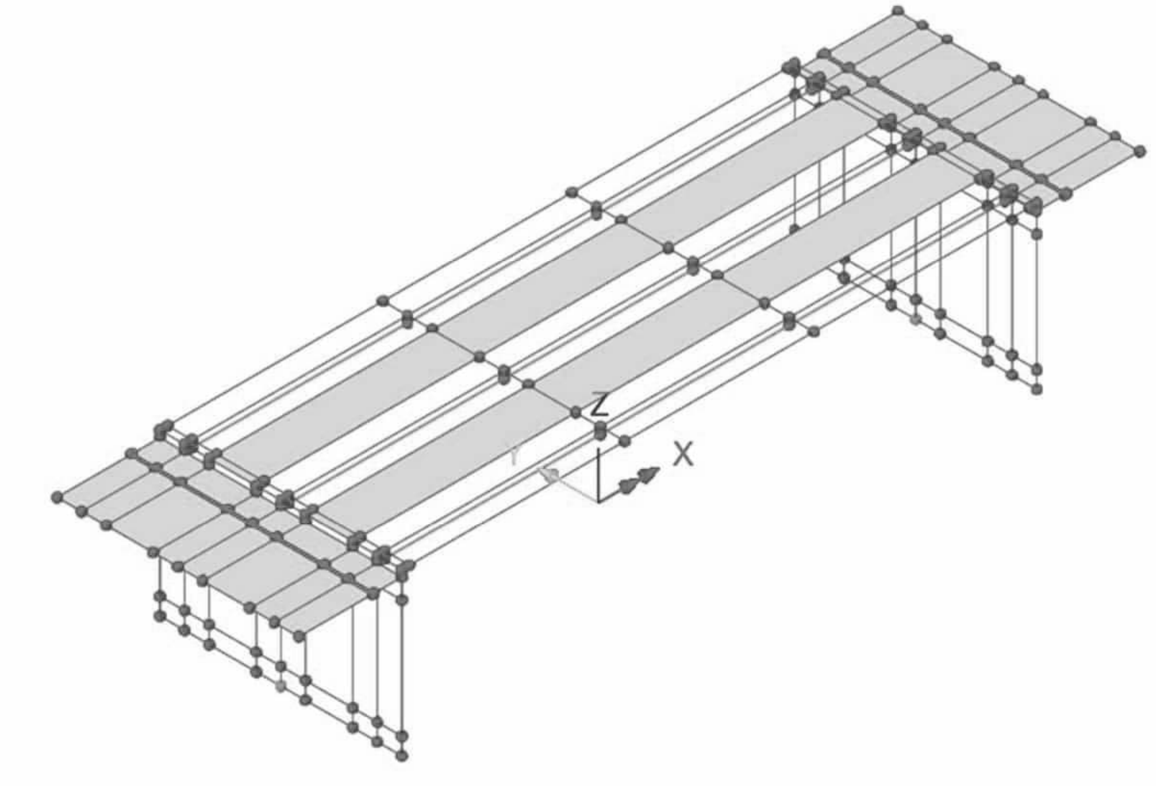


<u>Overview 3D</u>
Applied to beams in superstructure.

2.4.2.3 Material : Isotropic concrete C40/50 reduced ($0.6E_{ck}$)

$E = 0.6 \cdot E_{ck} = 0.6 \cdot 35 \cdot 10^3$ MPa $= 21.0 \cdot 10^3$ MPa



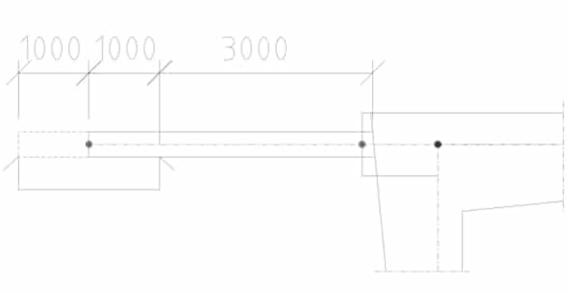


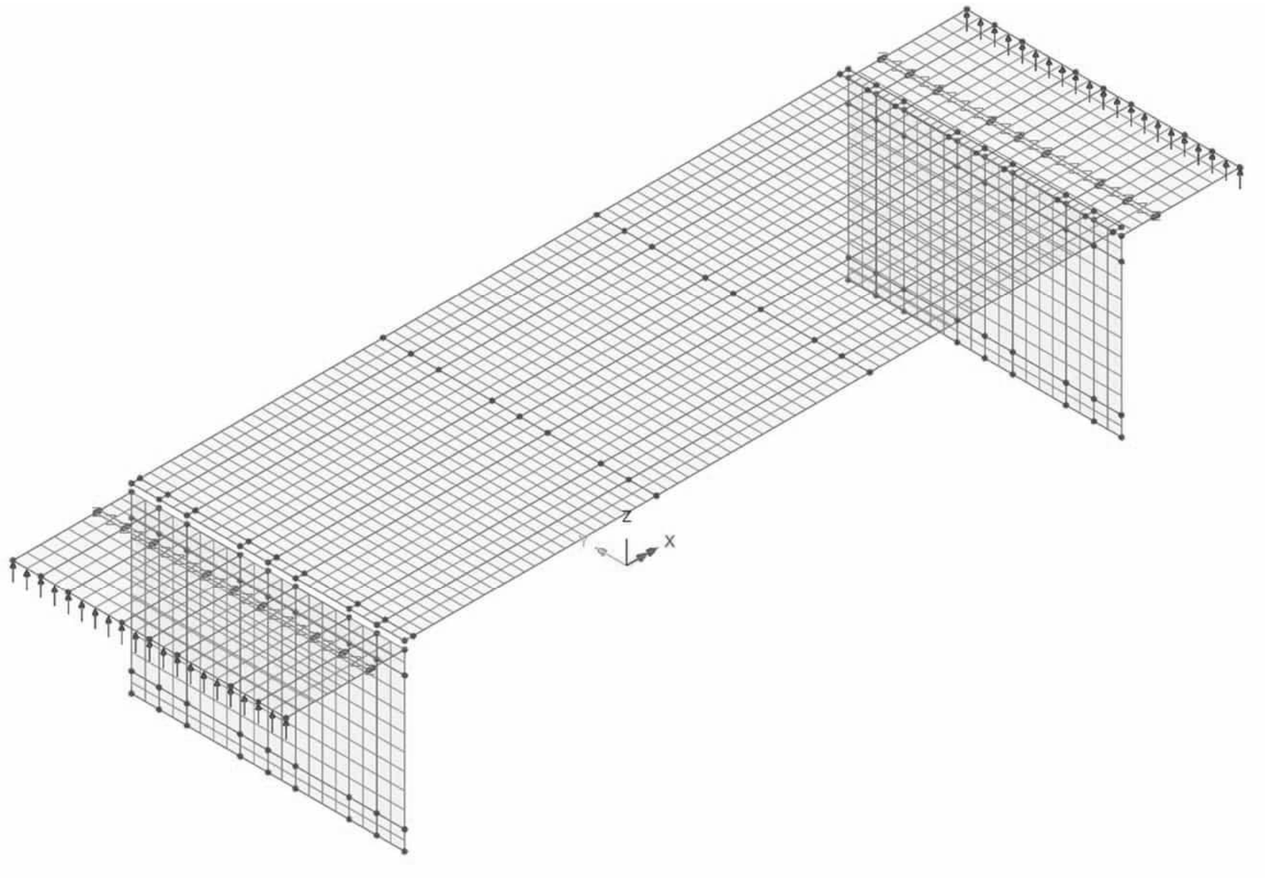
<u>Overview 3D</u>
Applied to deck between beams and link slab.

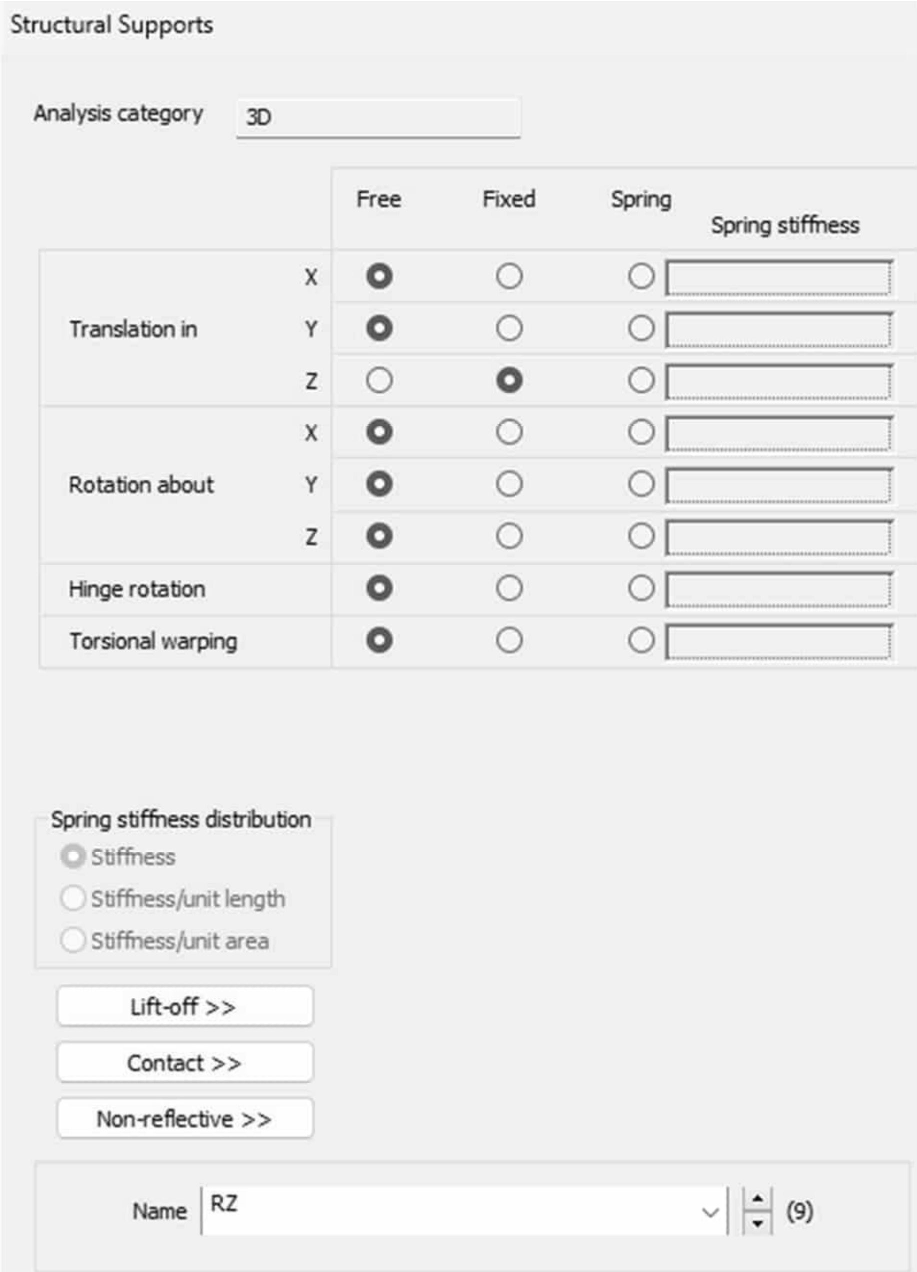
2.5 BOUNDARY CONDITIONS

2.5.1 Boundary conditions link slab

At a distance 1 m from edge of link slab att fictive line support is added in z-direction.





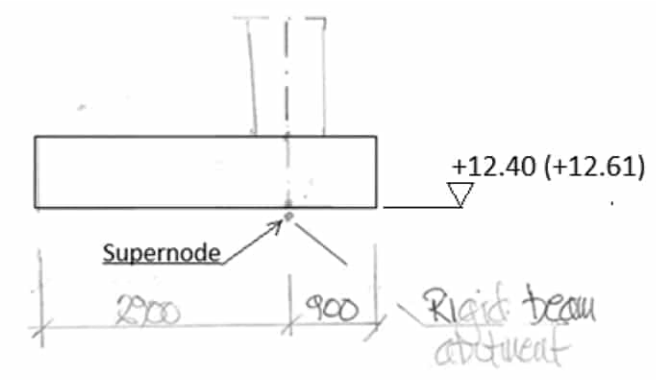


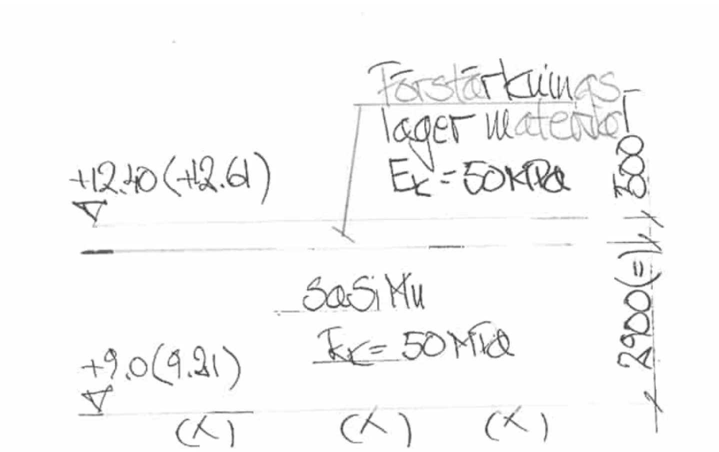
Structural Supports
Analysis category
3D
Free
Fixed
Spring
Spring stiffness
Translation in
X
Y
Z
Rotation about
X
Y
Z
Hinge rotation
Torsional warping
Spring stiffness distribution
Stiffness
Stiffness/unit length
Stiffness/unit area
Lift-off >>
Contact >>
Non-reflective >>
Name
RZ
(9)

2.5.2 Boundary conditions abutments

Boundary conditions for each support is modelled using super nodes.

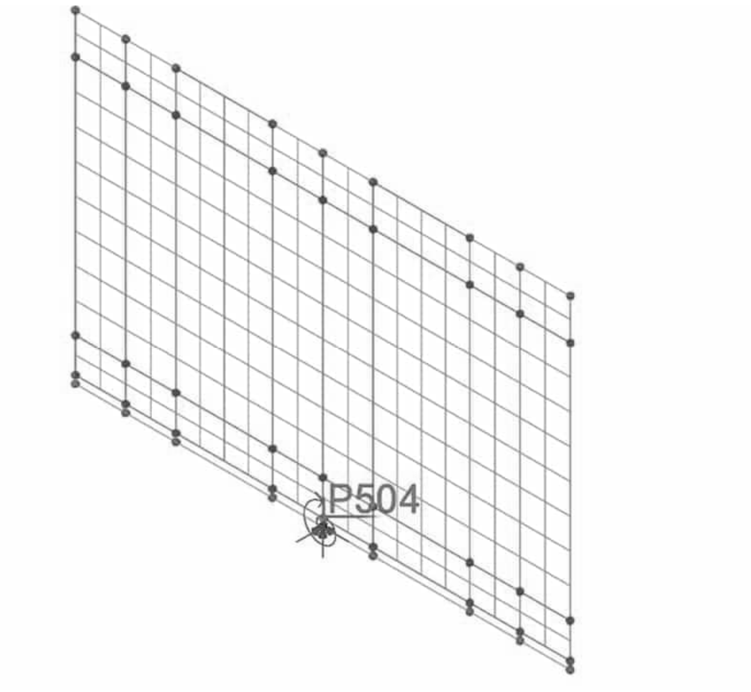
The super nodes are location at centre of rigid beam abutment, see sketches below.



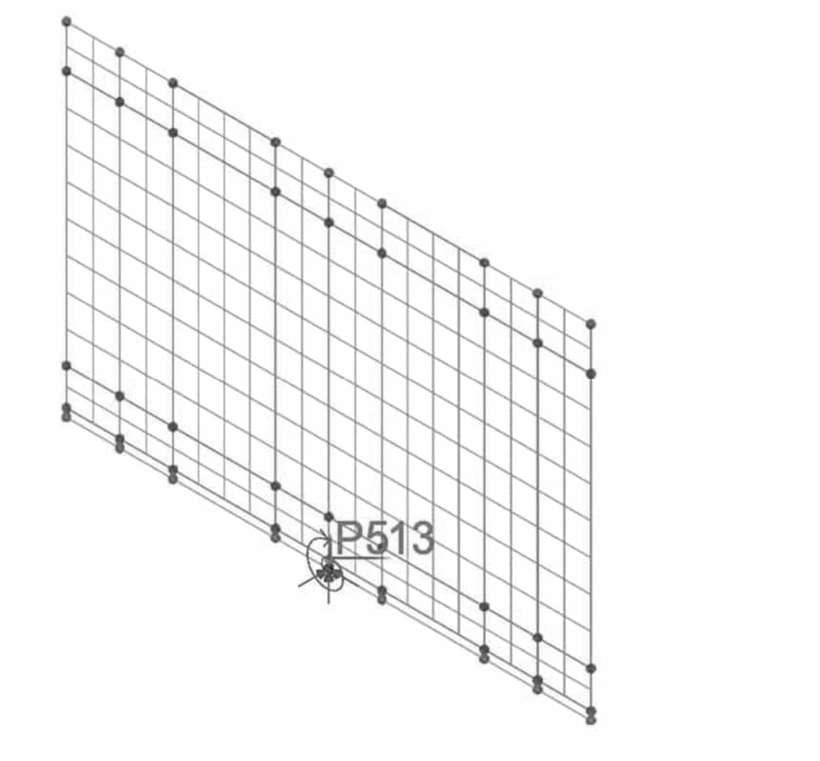


Geotechnical section at support 1
() marks support 2

Supernode – point foundation 1:



Supernode – point foundation 2:



Rotational stiffness of foundation is determined using software PROG G3.005.

Since the distance to solid ground (H) is less than twice the width of the bottom plate (2B), the method according to TRVINFRA-00227 appendix B5.1 is not applicable. Instead, a derived method for cases where H < 2B is used.

Stiffness transversal direction (Rotation X-X) :

$K_{Rx} = 10955 \cdot 10^3 \frac{kNm}{rad}$: page A2:46

Stiffness longitudinal direction (Rotation Y-Y) :

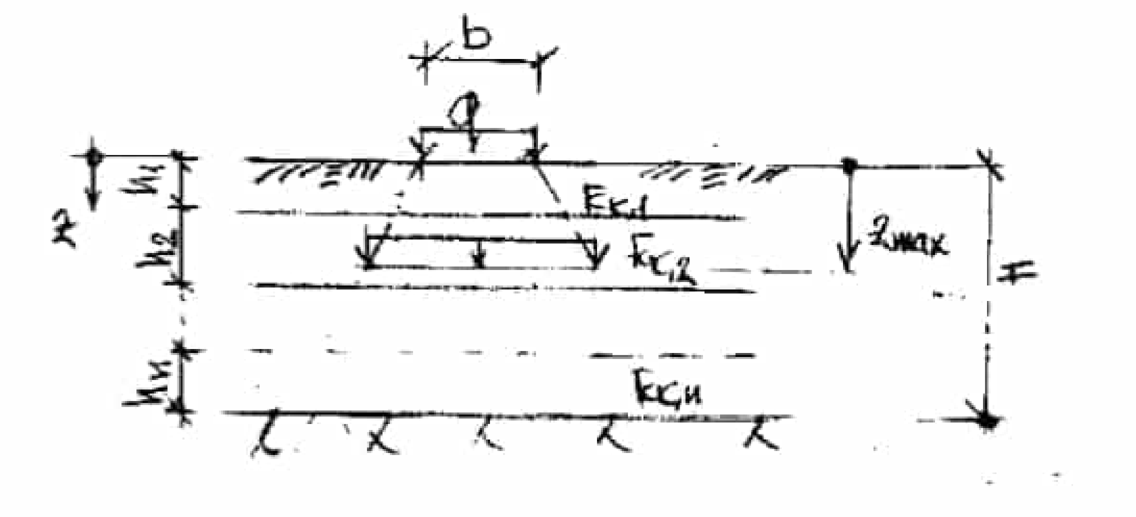
$K_{Ry} = 894 \cdot 10^3 \frac{kNm}{rad}$: page A2:46

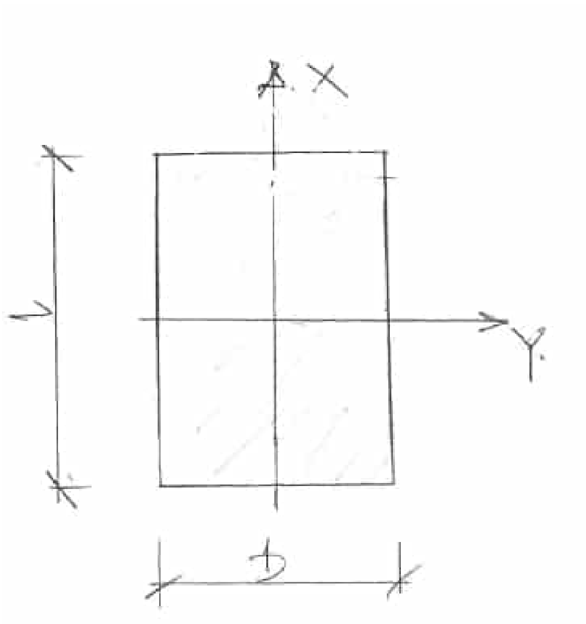
Object : Support 1/2

PRINCIPLE SKETCH

Geometry and foundation

The calculation of the equivalent stiffness in the foundation has been carried out according to TRVINFRA-00227, Appendix 4 (method 1). The equivalent E-modulus assumes a load spread of 2:1. The determination was made for a fictitious load corresponding to q (= 100 kPa).

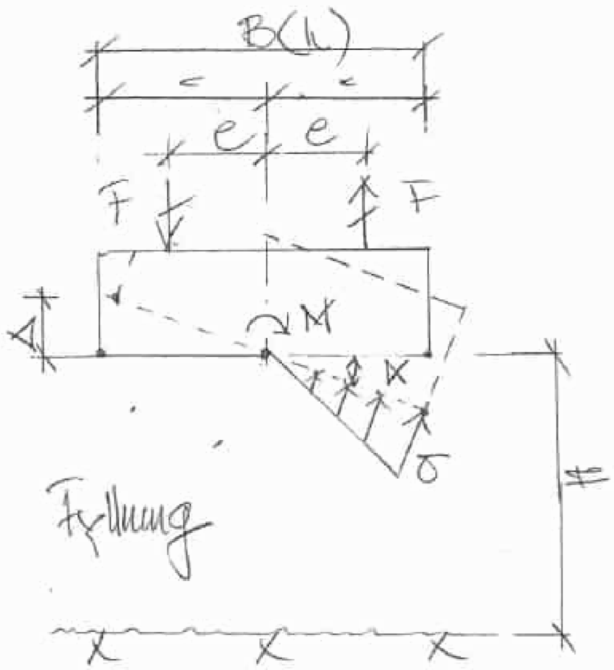




THEORY

When the distance to the rock is less than 2B, the formula below is applied, which also appears in BH page 594 (section 6.4:22 Calculation model).

When distance to rock is greater than 2B use TRVINFRA-00227, Appendix 5.



$$\Delta = \alpha \cdot \frac{B}{2} ; \qquad \varepsilon = \frac{\Delta}{H} = \frac{\alpha \cdot \frac{B}{2}}{H}$$

$$\sigma = \varepsilon \cdot E'_k = \frac{E'_k \cdot \alpha \cdot B}{2 \cdot H}$$

$$F = \sigma \cdot \frac{B}{2} \cdot \frac{1}{2} \cdot L = \frac{E'_k \cdot \alpha \cdot B}{2 \cdot H} \cdot \frac{B \cdot L}{4} = \frac{E'_k \cdot \alpha \cdot B^2 \cdot L}{8 \cdot H}$$

$$M = F \cdot 2e ; \qquad e = \frac{B}{3} ;$$

$$M = F \cdot \frac{2 \cdot B}{3} = \alpha \cdot \frac{L \cdot B^3 \cdot E'_k}{12 \cdot H}$$

$$\Rightarrow \frac{\alpha}{M} = \frac{12 \cdot H}{L \cdot B^3 \cdot E'_k}$$

INPUT

Geometry

Bottom slab : $b := 3.80\ m$ $l := 13.30 \cdot m$

Soil material

Number of layers (min. 2 layers): $n := 2$

Layer	E_k	h
1	50	0,50
2	50	2,90
-	MPa	m

CALCULATIONS

Total layer thickness

$$H := \sum_{i=1}^{n} h_i \qquad H = 3.4\ m$$

Total settlement thickness

$$z_{max} := min(2 \cdot b, H) \qquad z_{max} = 3.4\ m$$

Levels for each layer

$$z_s := \begin{array}{l} z_1 \leftarrow 1 \cdot mm \\ \text{for } i \in 1..n \\ \quad z_{2 \cdot i} \leftarrow z_{2 \cdot i - 1} + h_i - 1 \cdot mm \\ \quad z_{2 \cdot i + 1} \leftarrow z_{2 \cdot i} + 1 \cdot mm \\ z_1 \leftarrow 0 \cdot mm \end{array}$$

$$z_s^{\mathrm{T}} = [0\ \ 0.5\ \ 0.501\ \ 3.4\ \ 3.401]\ m$$

Function - settlement modulus

$$E_{sk} := \begin{array}{l} E_1 \leftarrow E_{k_1} \\ \text{for } i \in 1..n-1 \\ \quad E_{2 \cdot i} \leftarrow E_{k_i} \\ \quad E_{2 \cdot i + 1} \leftarrow E_{k_{i+1}} \\ E_{2 \cdot n} \leftarrow E_{k_n} \\ E_{2 \cdot n + 1} \leftarrow 1000 \cdot MPa \end{array}$$

$$E_{sk}^{\mathrm{T}} = [50\ \ 50\ \ 50\ \ 50\ \ 1000]\ MPa$$

$$E_{kar}(z) := \text{linterp}(z_s, E_{sk}, z)$$

Stress according to method 2:1

$q := 100 \cdot kPa$

$$\Delta\sigma_v(z) := q \cdot \frac{b \cdot l}{(b+z) \cdot (l+z)}$$

Characteristic settlement

$$s_k := \int_{0 \cdot m}^{H} \frac{\Delta\sigma_v(z)}{E_{kar}(z)} \, dz$$

$s_k = 4.4 \ mm$

Equivalent settlement modulus

$$E'_k := \frac{\int_{0 \cdot m}^{z_{max}} \Delta\sigma_v(z) \, dz}{s_k}$$

$E'_k = 50 \ MPa$

Function - stiffness when H < 2B
(See derivation section THEORY)

$$k_{\theta k}(B, L) := \frac{L \cdot B^3 \cdot E'_k}{12 \cdot H}$$

,

RESULTS

Results when H < 2B

Rotation about bottom slab short direction (x-x direction):

$k_{\theta k}(b, l) = 894360\ \frac{kNm}{rad}$

$C_{\phi.1} := \frac{1}{k_{\theta k}(b, l)}$

$10^9 \cdot C_{\phi.1} = 1118\ \frac{rad}{kNm}$

Rotation about bottom slab long direction (y-y direction):

$k_{\theta k}(l, b) = 10955908\ \frac{kNm}{rad}$

$C_{\eta.1} := \frac{1}{k_{\theta k}(l, b)}$

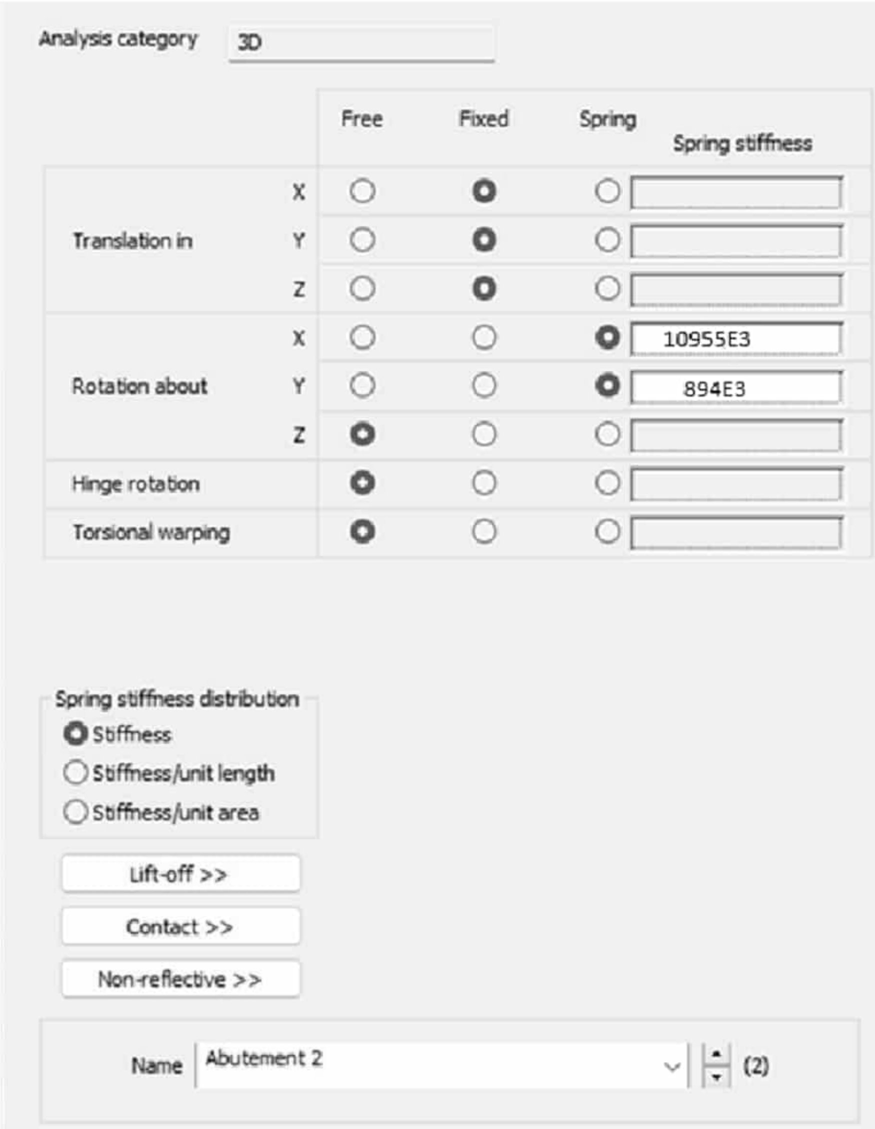
$10^9 \cdot C_{\eta.1} = 91\ \frac{rad}{kNm}$

2.5.1 Boundary support 1

Supernode at support 1 is modelled as seen below. This super node is termed point P429 in model of system analysis.

2.5.2 Boundary support 2

Supernode at support 2 is modelled as seen below. This super node is termed point P430 in model of system analysis.



2.6 MESH

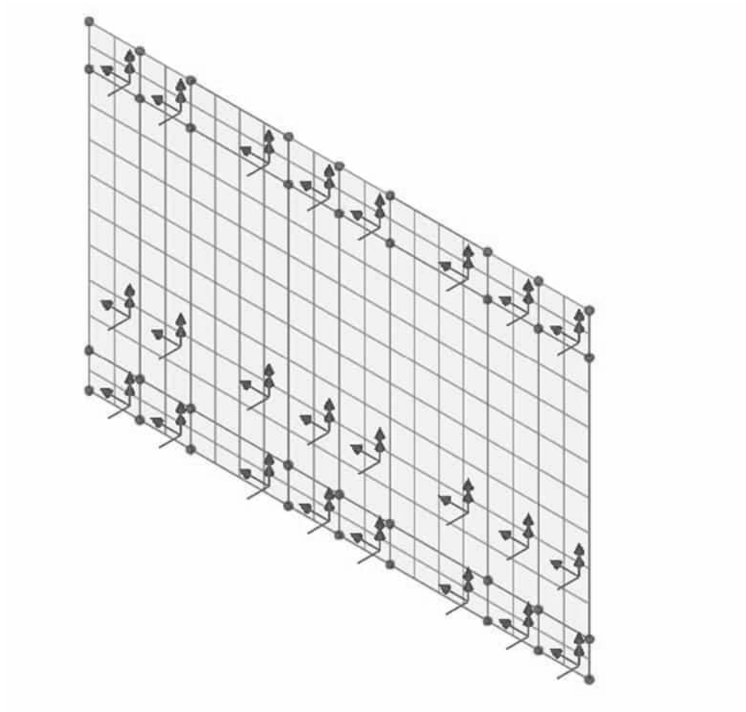
2.6.1 Shell element (QTS4): linear

Bridge deck is model using shell elements.

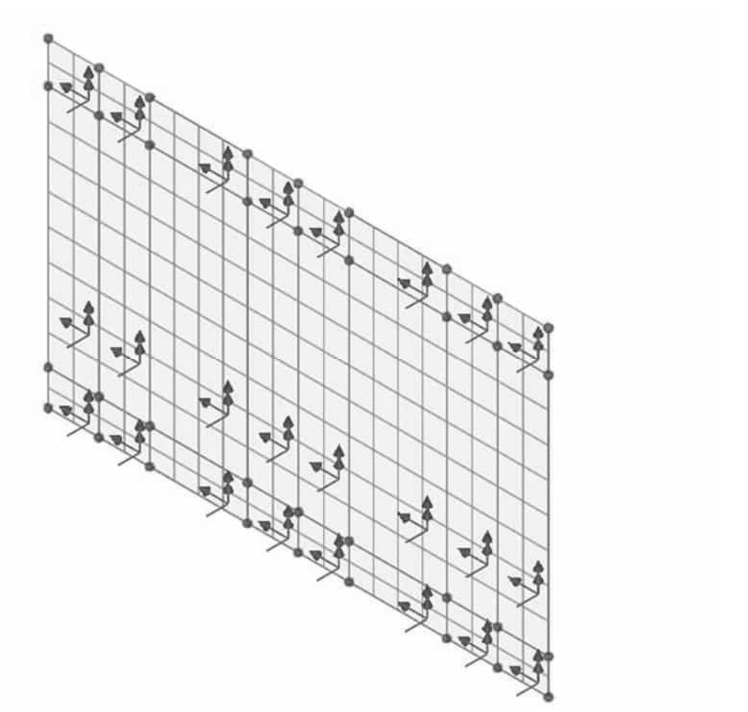
Shell elements are modelled with various subdivisions as seen below.

Type	x-divisions	y-divisions
Element 20 x 2	20	2
Element 20 x 4	20	4
Element 24 x 2	24	2
Element 24 x 4	24	4
Element 1 x 2	1	2
Element 1 x 4	1	4
Element 3 x 4	3	4
Element 3 x 2	3	2
Element 6 x 2	6	2
Element 6 x 4	6	4
Element 8 x 2	8	2
Element 8 x 4	8	4
Element 2 x 2	2	2
Element 2 x 4	2	4

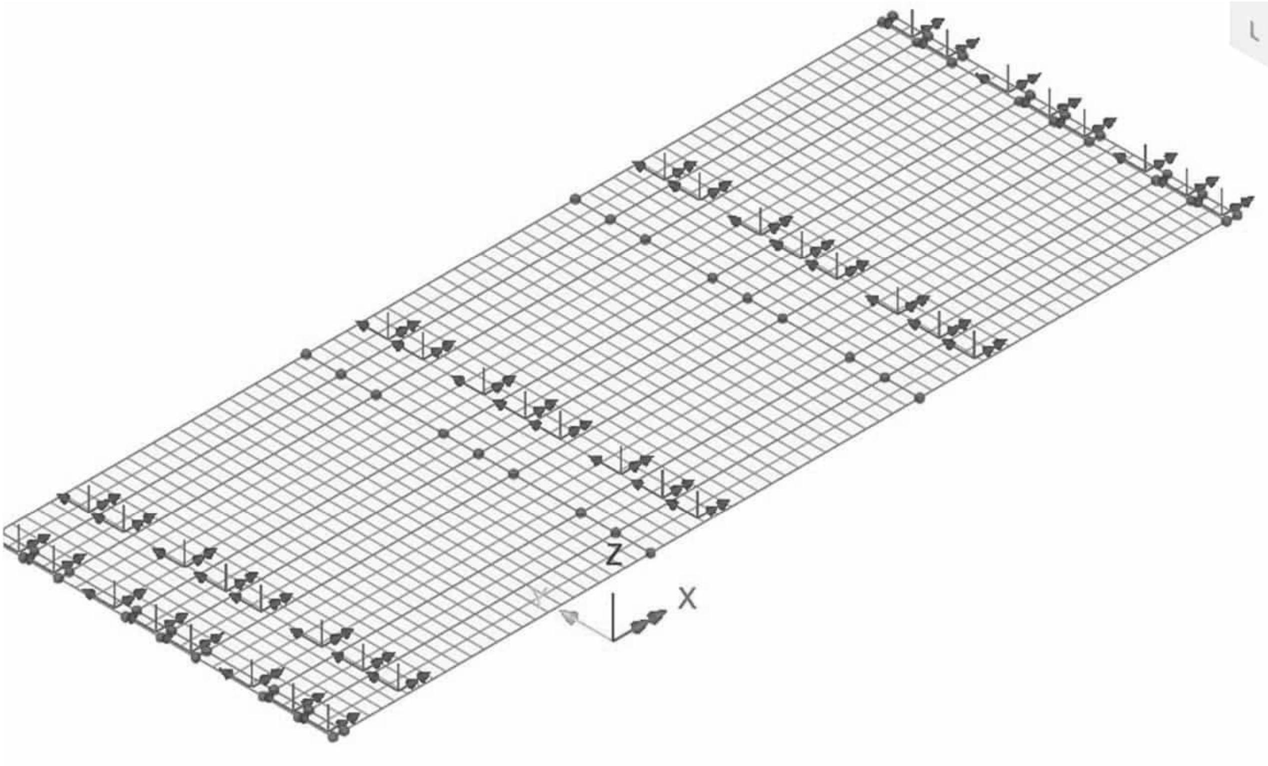
Abutment 1 :



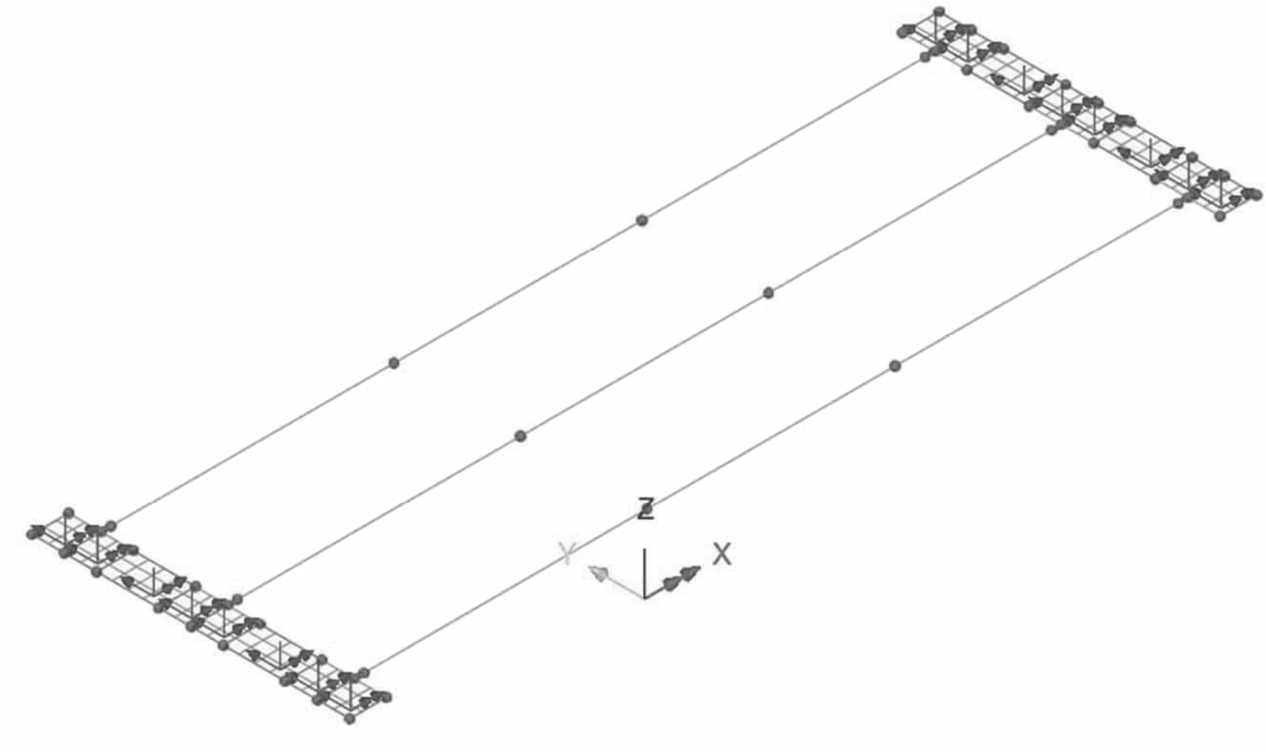
Abutment 2 :



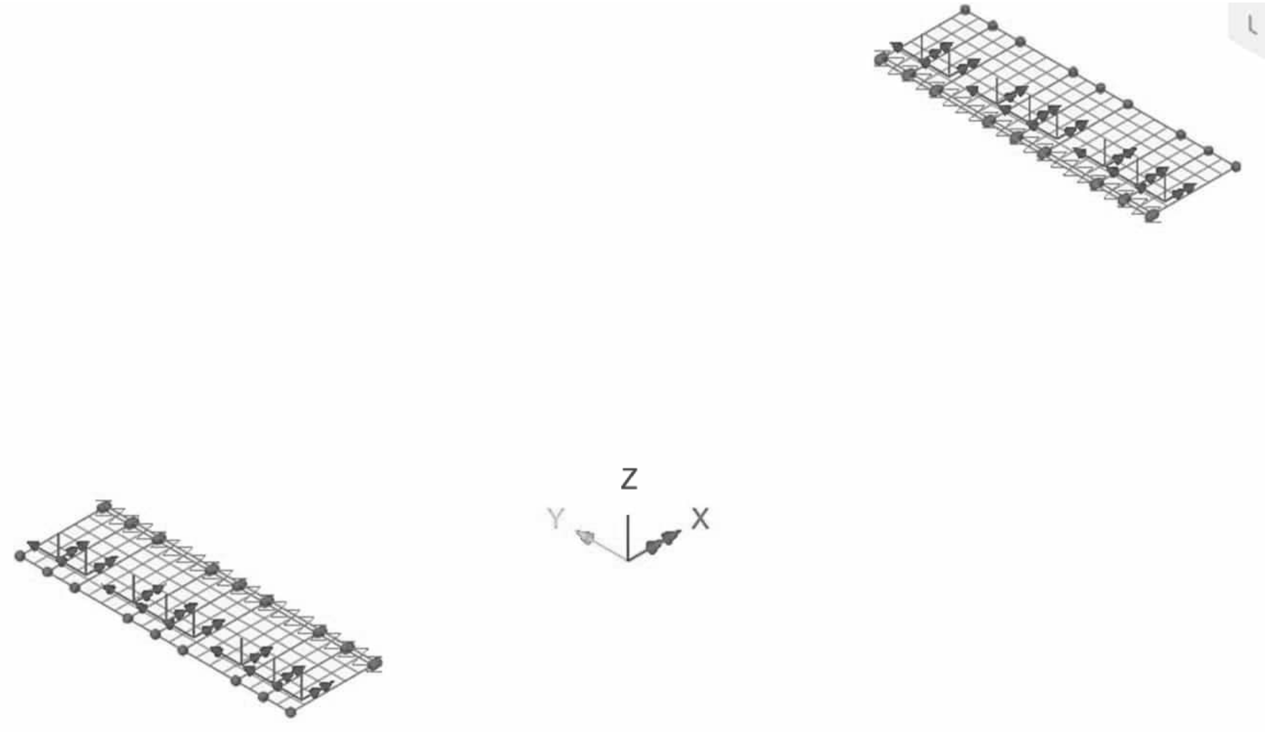
Deck - superstructure:



Beams - superstructure:



Link slabs - superstructure:



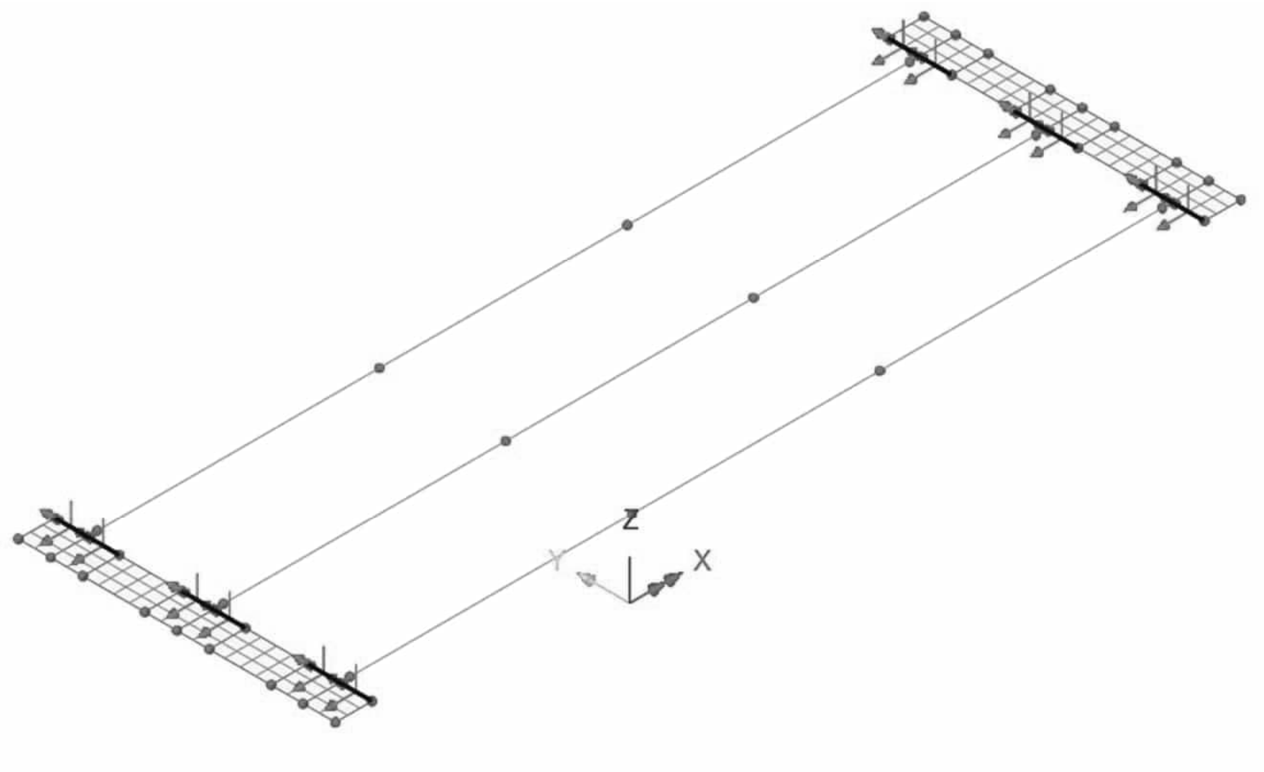
2.6.2 Beam element (BMI21) : linear

Longititudinal beams in superstructure are modelled using beam elements.

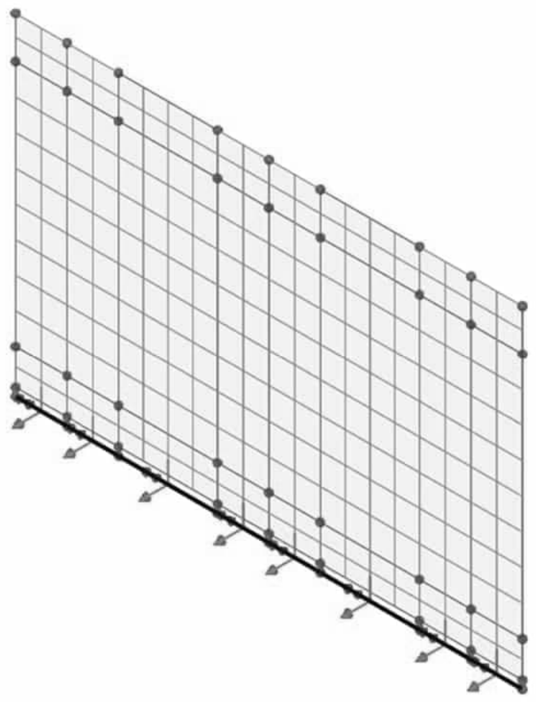
Beams elements are modelled with various subdivisions as seen below.

Typ	Divisions	End release: Start	End release: End
Element 1	1	None	None
Element 20	20	None	None
Element 24	24	None	None
Element 2	2	None	None
Element 4	4	None	None

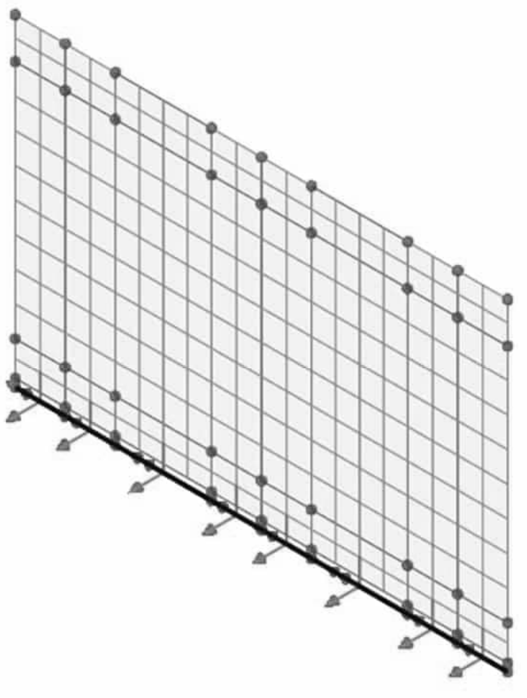
Rigid beam - superstructure :



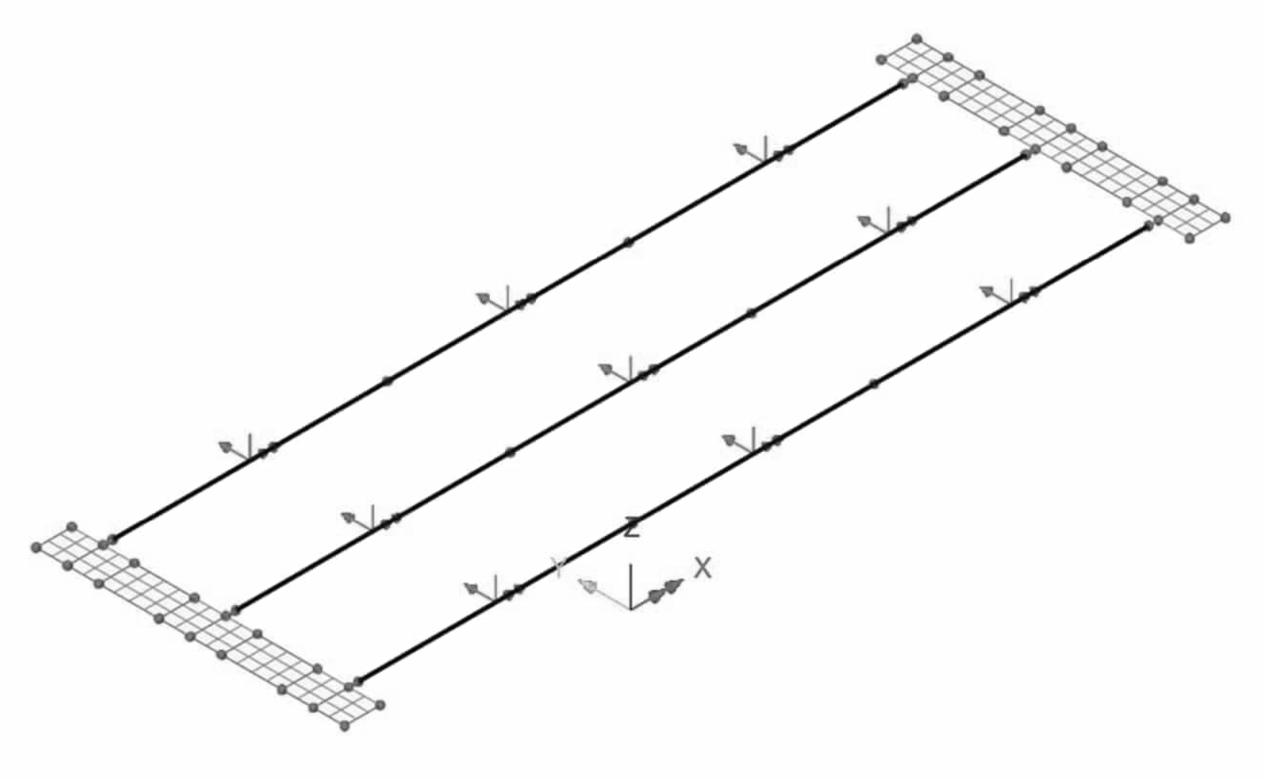
Rigid beam abutment 1:



Rigid beam abutment 2:



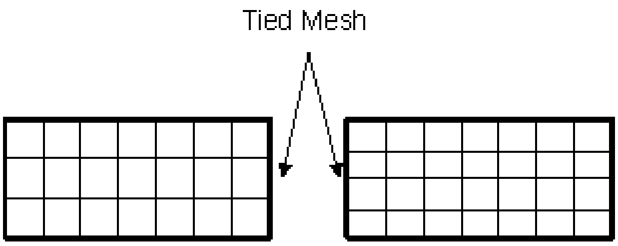
Beam - superstructure :



2.6.3 Rigid connections (Tied Mesh)

Tied mesh are used to two different locations.

Connection Tied Mesh: Rigid constraint

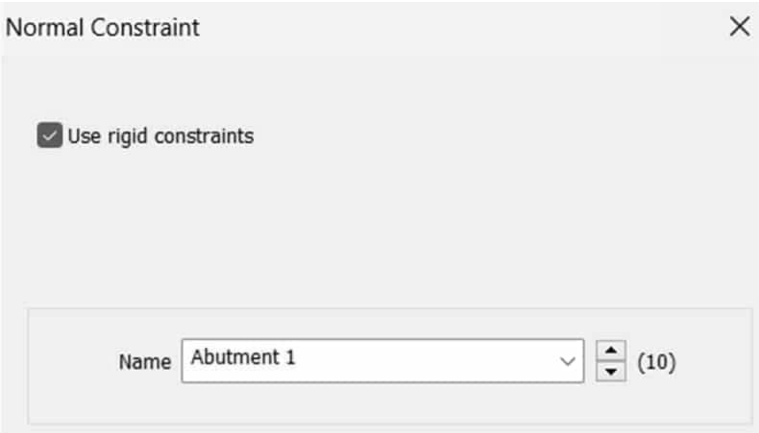


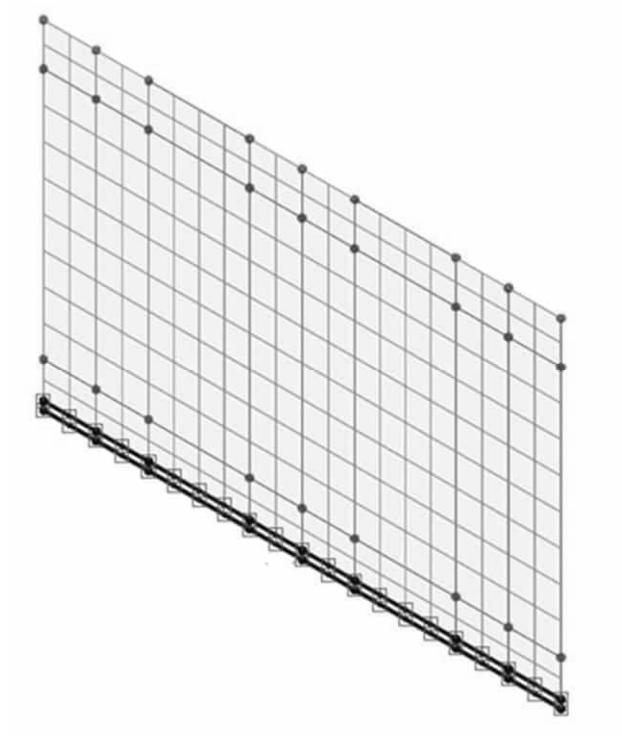
2.6.3.1 Tied mesh abutments

Type of node at rigid beams: Master

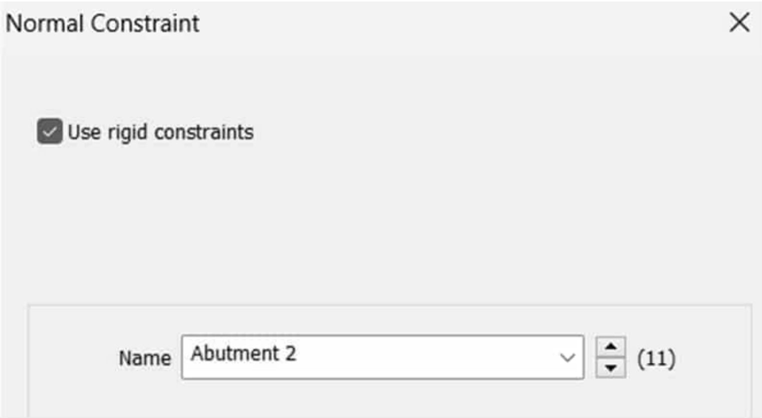
Type of node at bottom of abutments: Slave

Abutment 1:

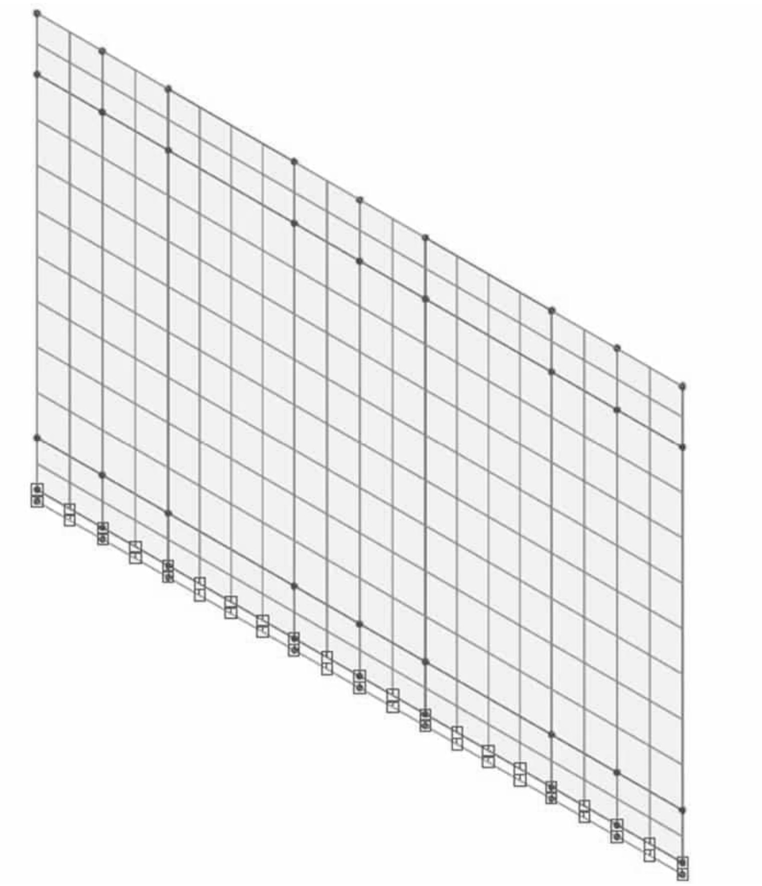




Abutment 2:



Normal Constraint
Use rigid constraints
Name
Abutment 2
(11)



2.6.3.2 Tied mesh superstructure

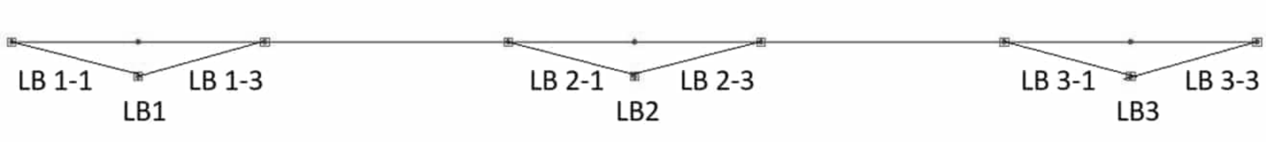
Deck is modelled as shell elements. They are defined by nodal surface.

Ribs are modelled as rectangular beam elements. They are defined by nodal lines.

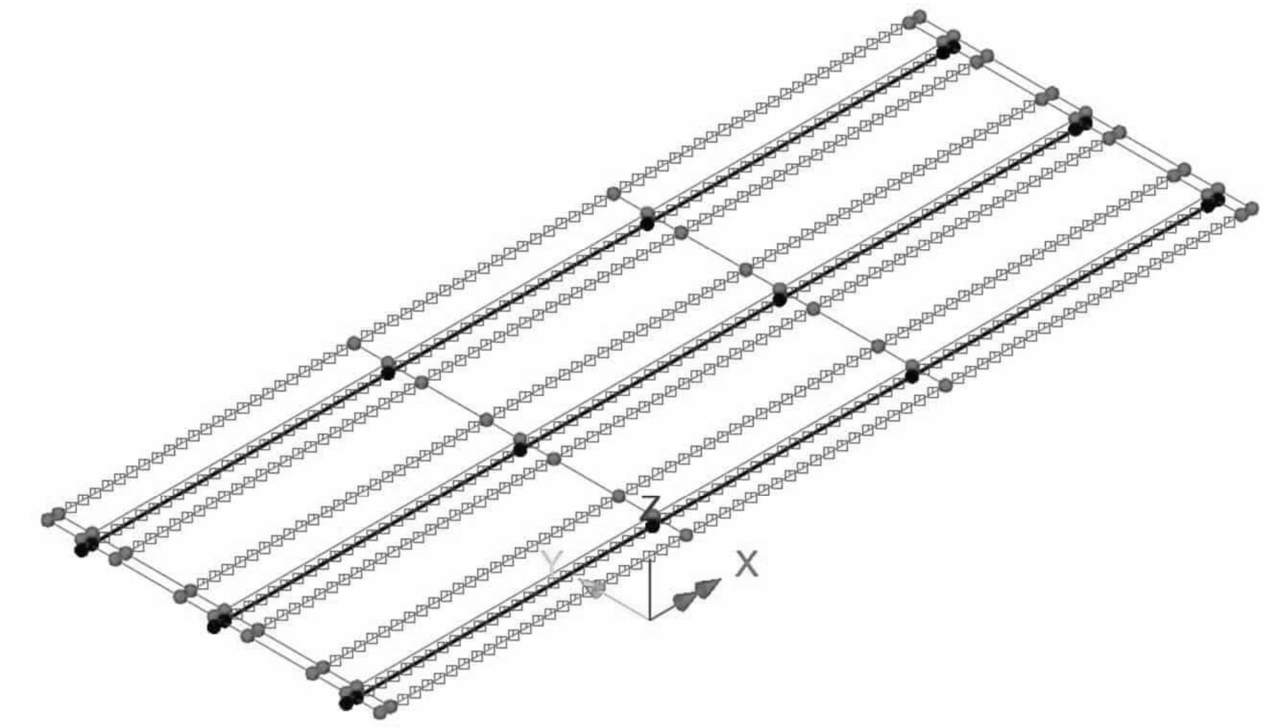
Deck is tied to ribs using constant constraints.

Deck is defined as master nodes.

Load cases excluding shrinkage & temperature (*Analysis 1*):

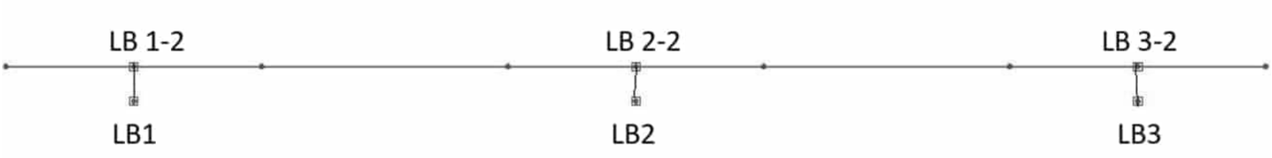


Cross section

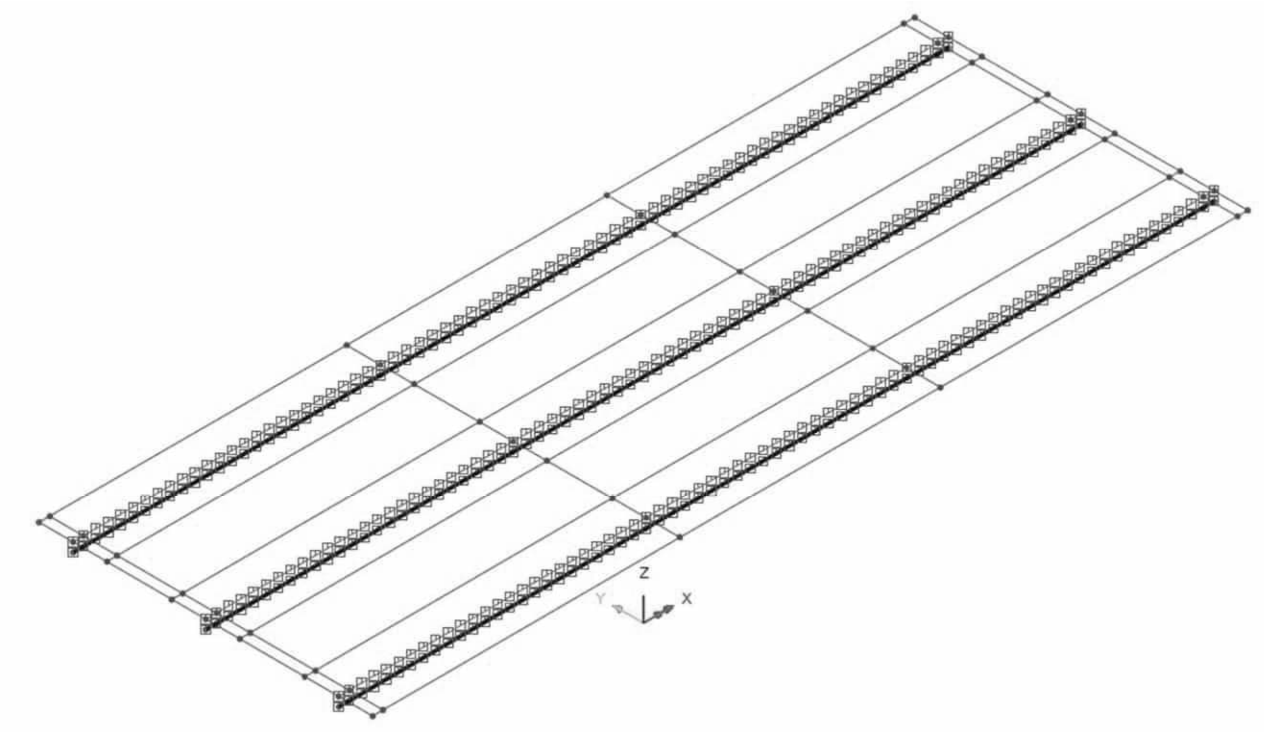


Overview 3D

Load cases shrinkage & temperature (*Analysis 2*):



Cross section

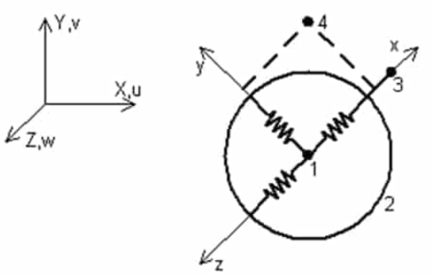


Overview 3D

2.6.4 Joint element between lines (JNT4) : linear

Connection between link slab & superstructure is of type joint element, see presentation below. The joint uses no rotational stiffness in order achieve a hinge.

Element Name	**JNT4**



Element Group	Joints
Element Subgroup	3D Joints
Element Description	A 3D joint element which connects two nodes by three springs in the local x, y and z-directions.
Number Of Nodes	4. The 3rd and 4th nodes are used to define the local x-axis and local xy-plane.
Freedoms	U, V, W: at nodes 1 and 2 (active nodes).
Node Coordinates	X, Y, Z: at each node.

Material:

Analysis category: 3D

Assignment to: Lines

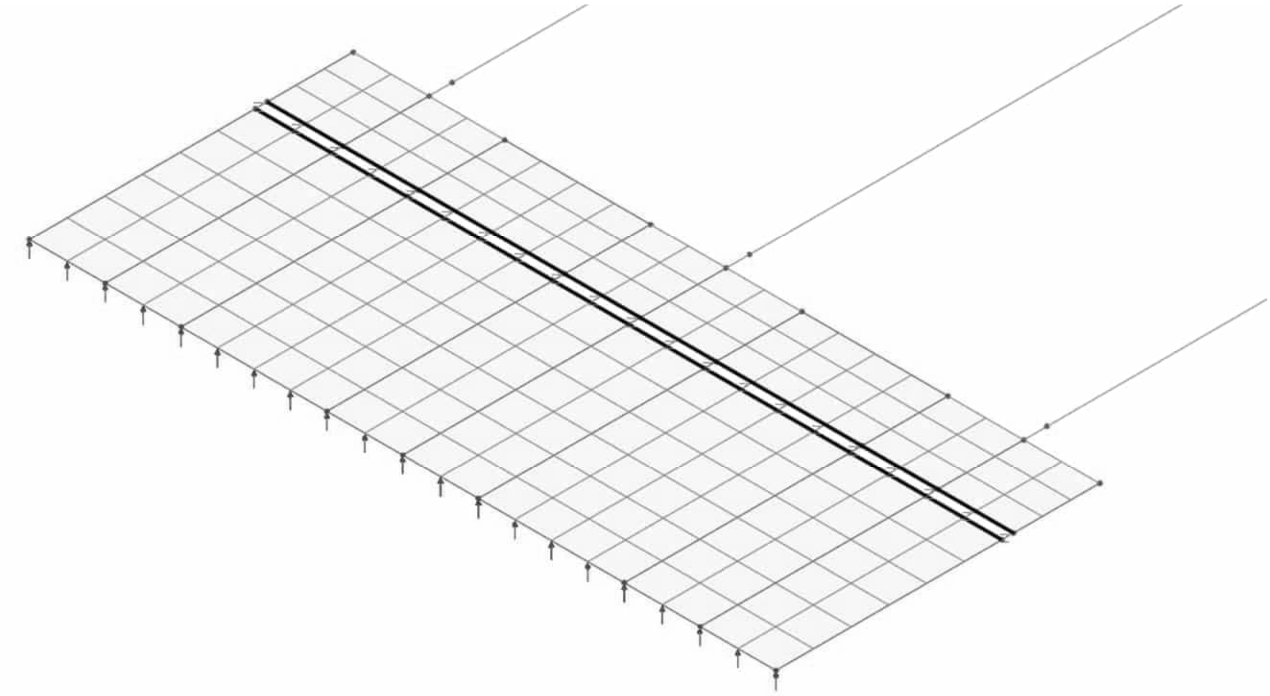
Joint type: Joint no rotational stiffness

Properties specified for each freedom

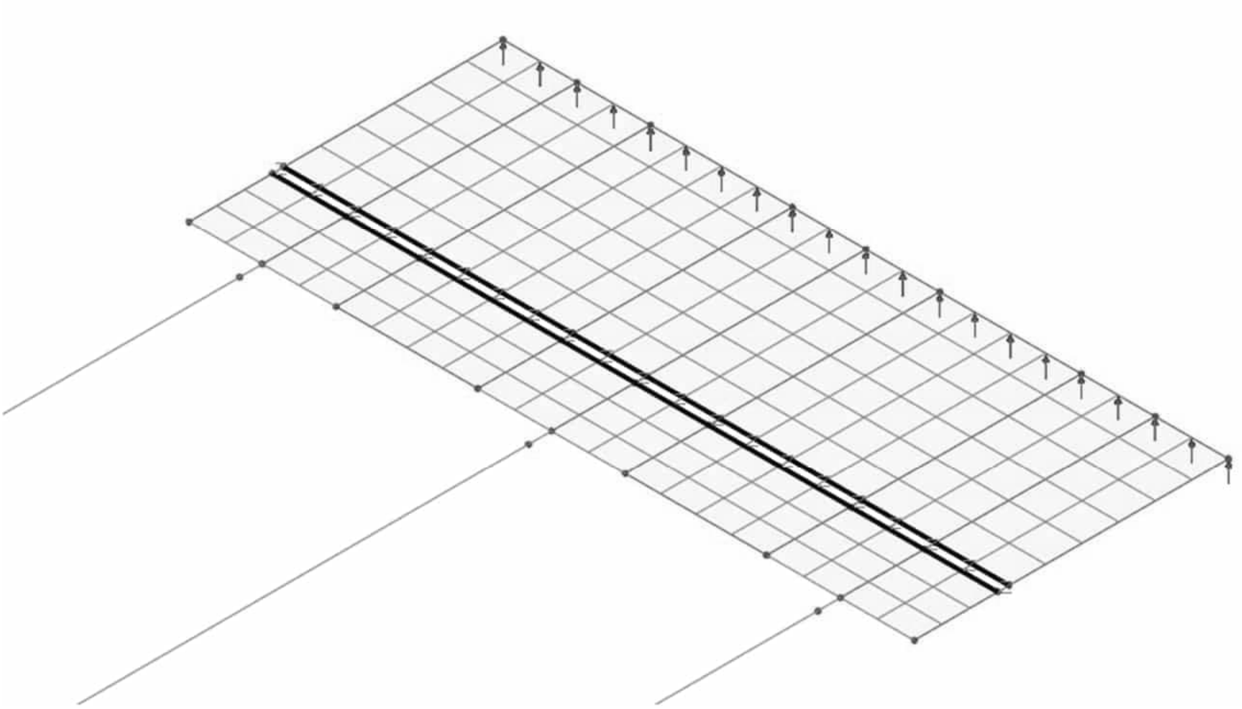
	u	v	w
Elastic spring stiffness	1,0E12	1,0E12	1,0E12

Name: Joint material JT4 (3)

Link slab at Abutment 1:



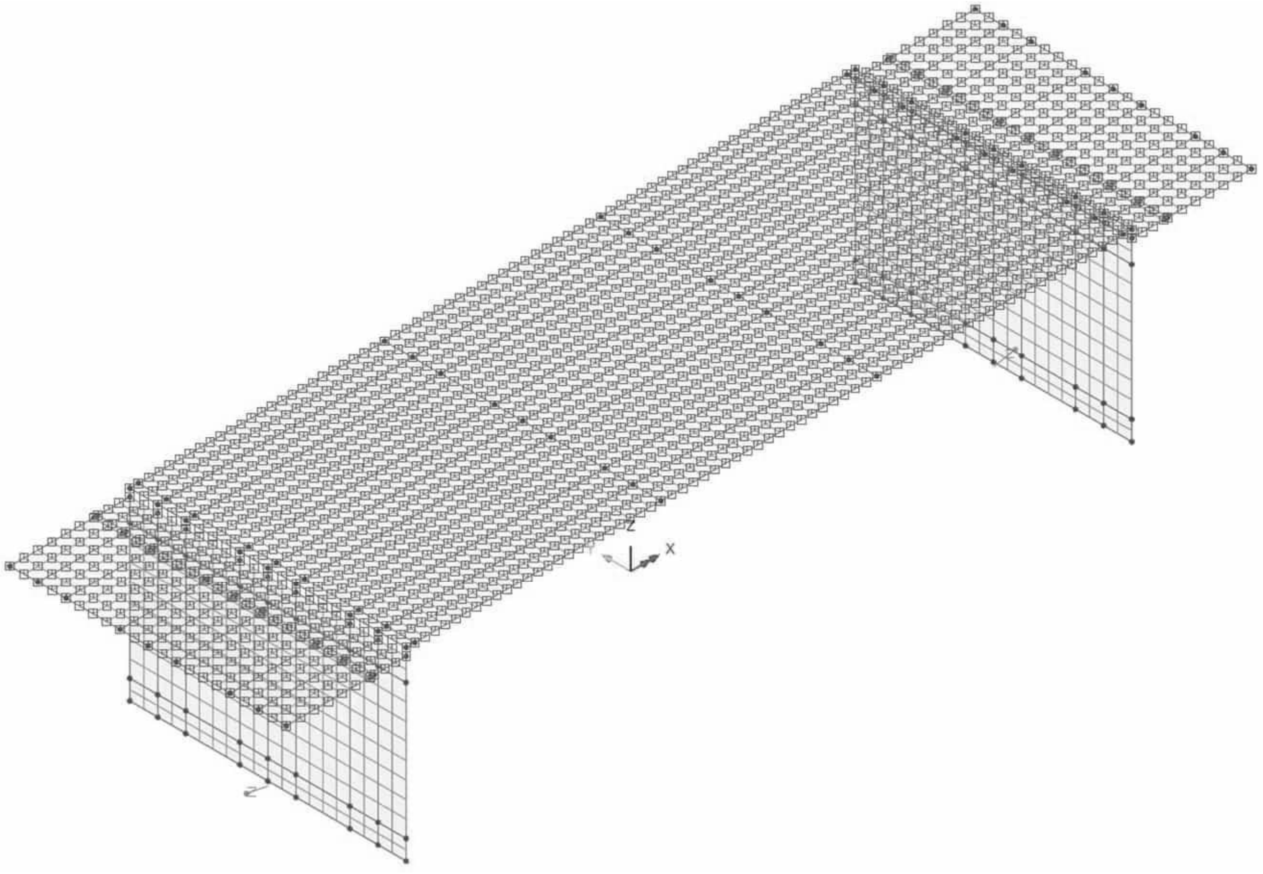
Link slab at Abutment 2:



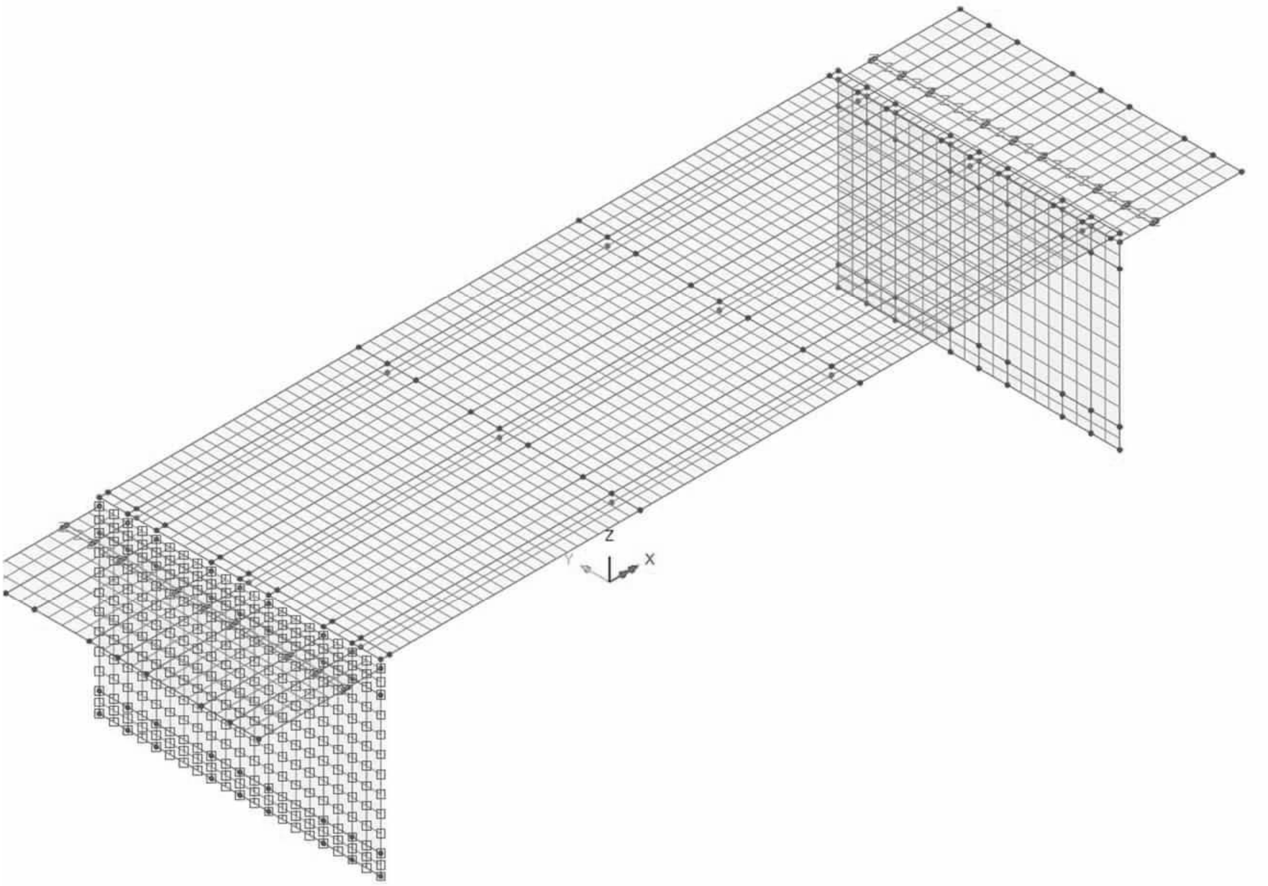
2.7 SEARCH AREA

Discrete load can be applied to structure as geometrical load areas. In FEM-program load areas are termed Search Area.

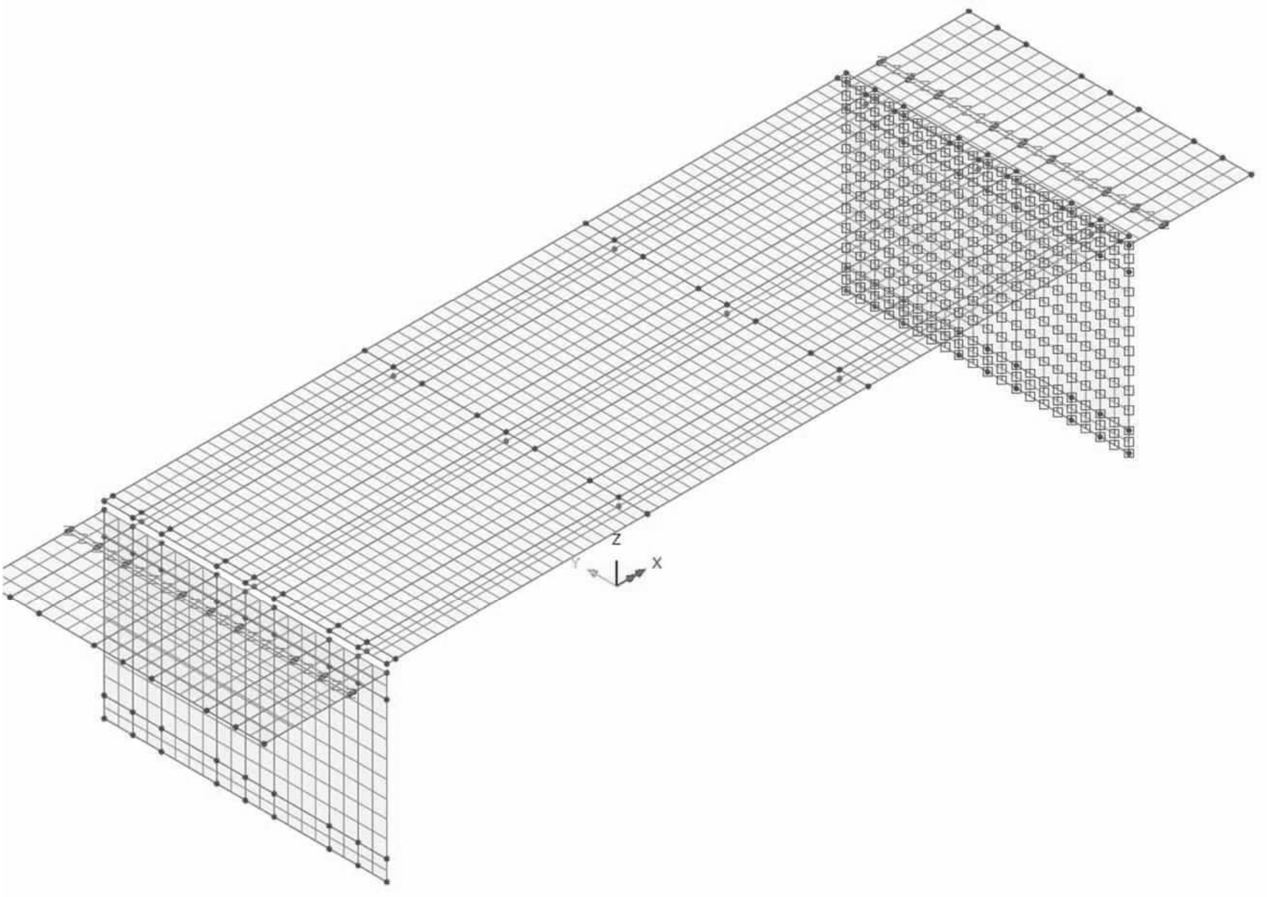
2.7.1 Search area : Superstructure



2.7.2 Search area : Abutment 1



2.7.3 Search area : Abutment 2

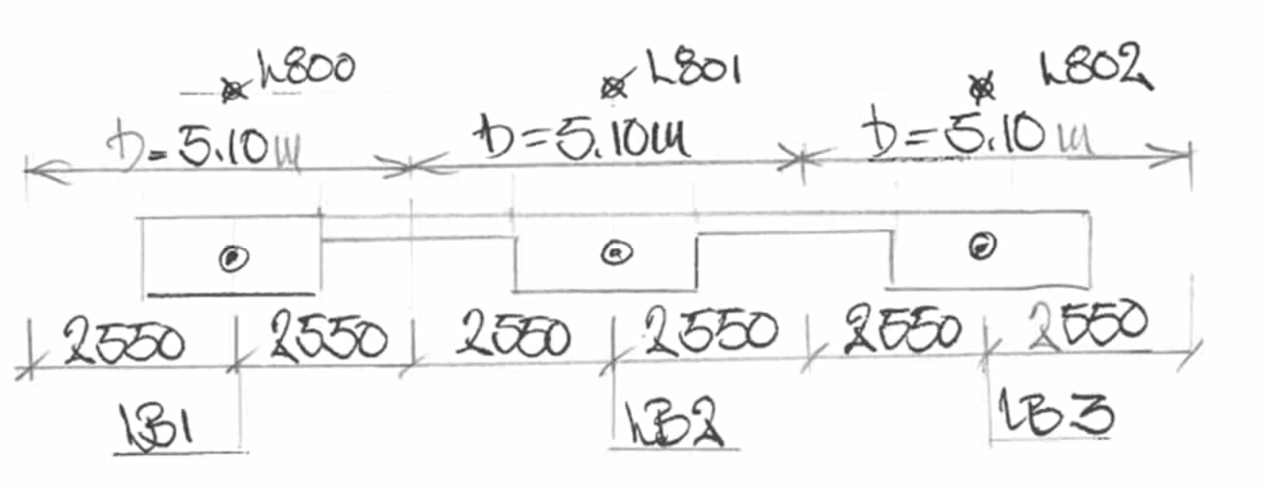


2.8 SLICE RESULTANTS BEAMS/SHELLS

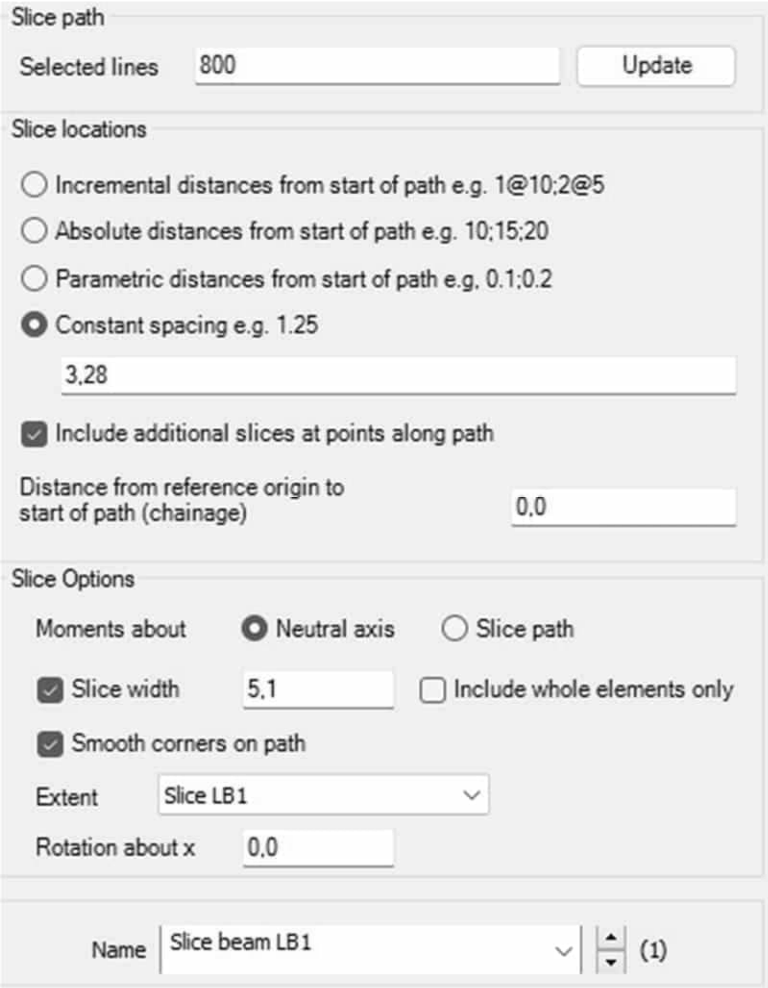
Equivalent section forces will be determined at 8 sub-points for each main beam. This is done by studying load effects in the Nodal surface and Nodal line for the respective main beams LB 1 and LB 2.

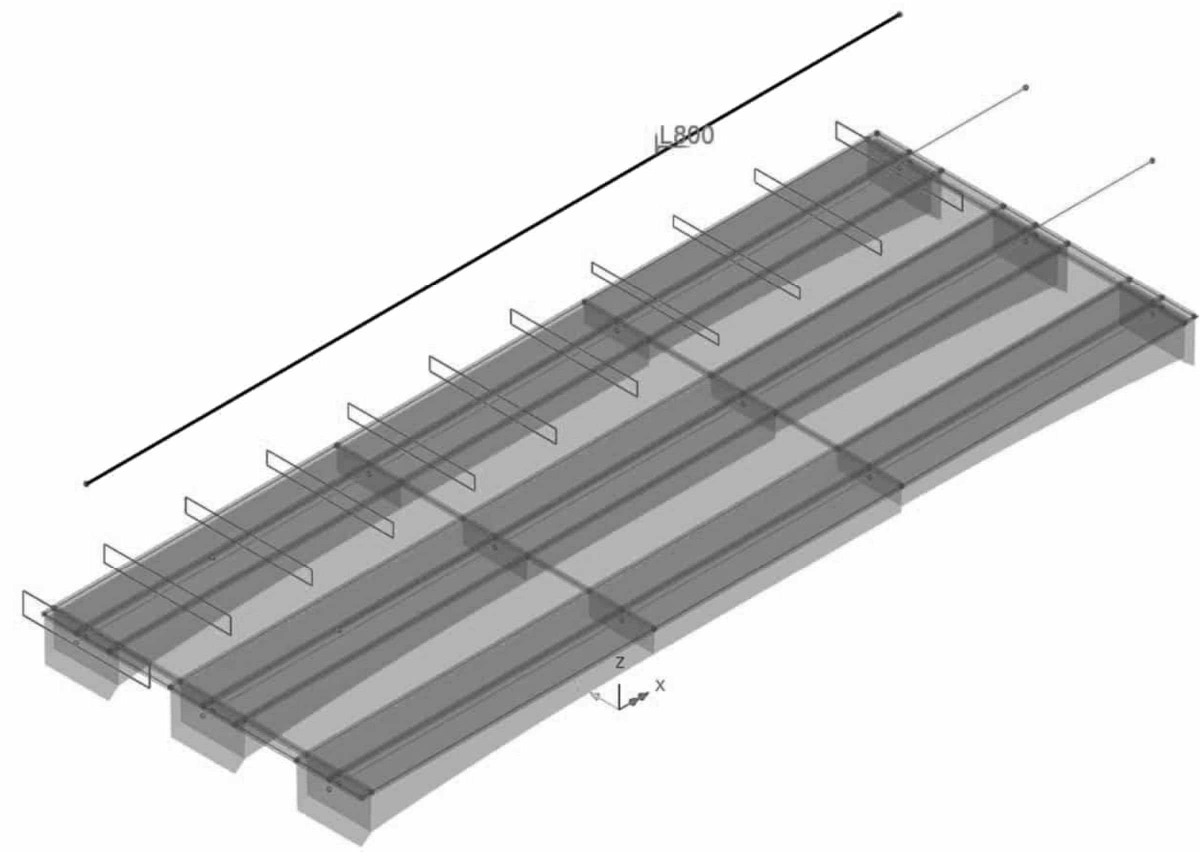
FEM-program has a script called "Slice Resultant Beams/Shells" to handle this, see the presentation below.

Beam	Path line	Extent	Remark
LB1	800	Slice LB1	Width = 5.10 m
LB2	801	Slice LB2	Width = 5.10 m
LB3	802	Slice LB3	Width = 5.10 m
-	-	-	-



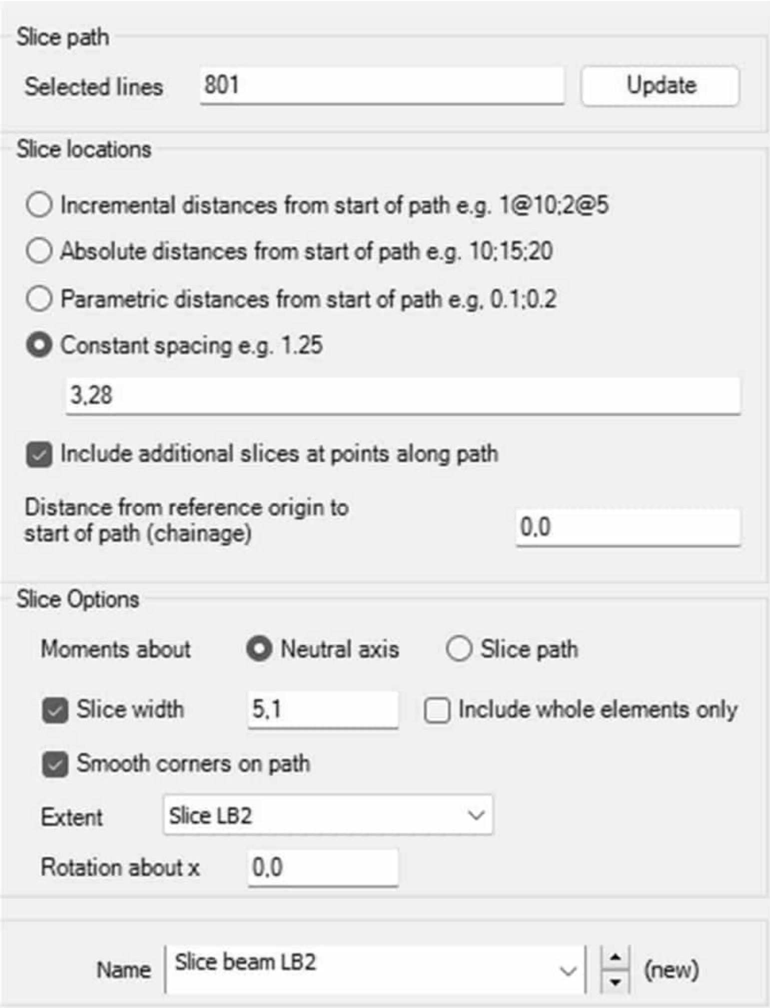
2.8.1 Slice beam LB1

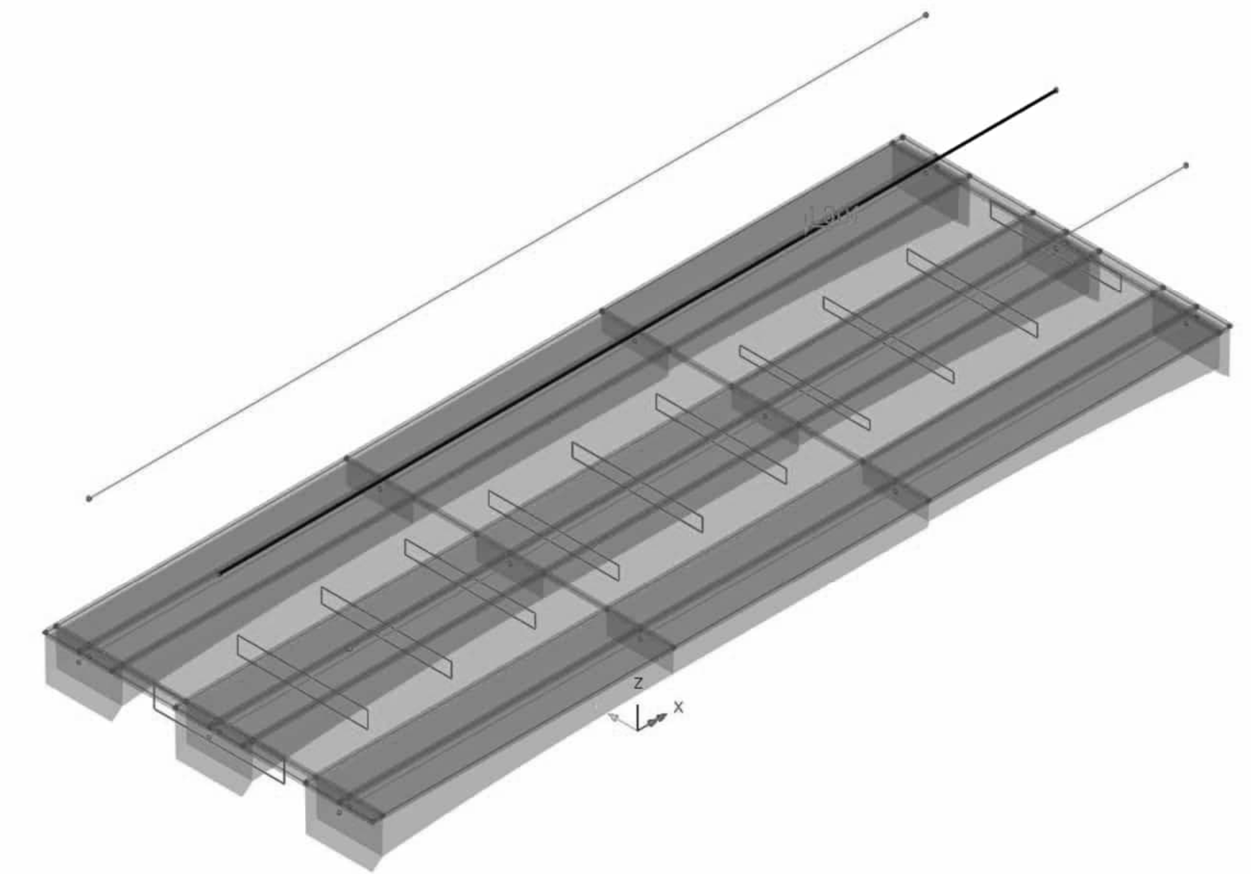




OVERVIEW

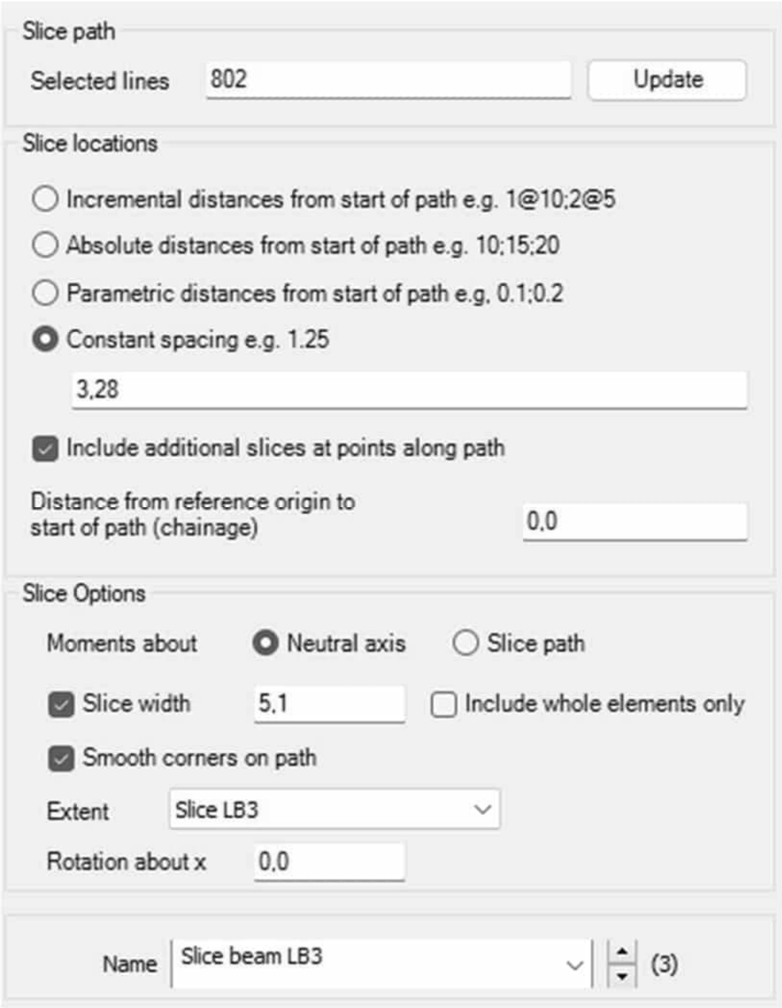
2.8.2 Slice beam LB2

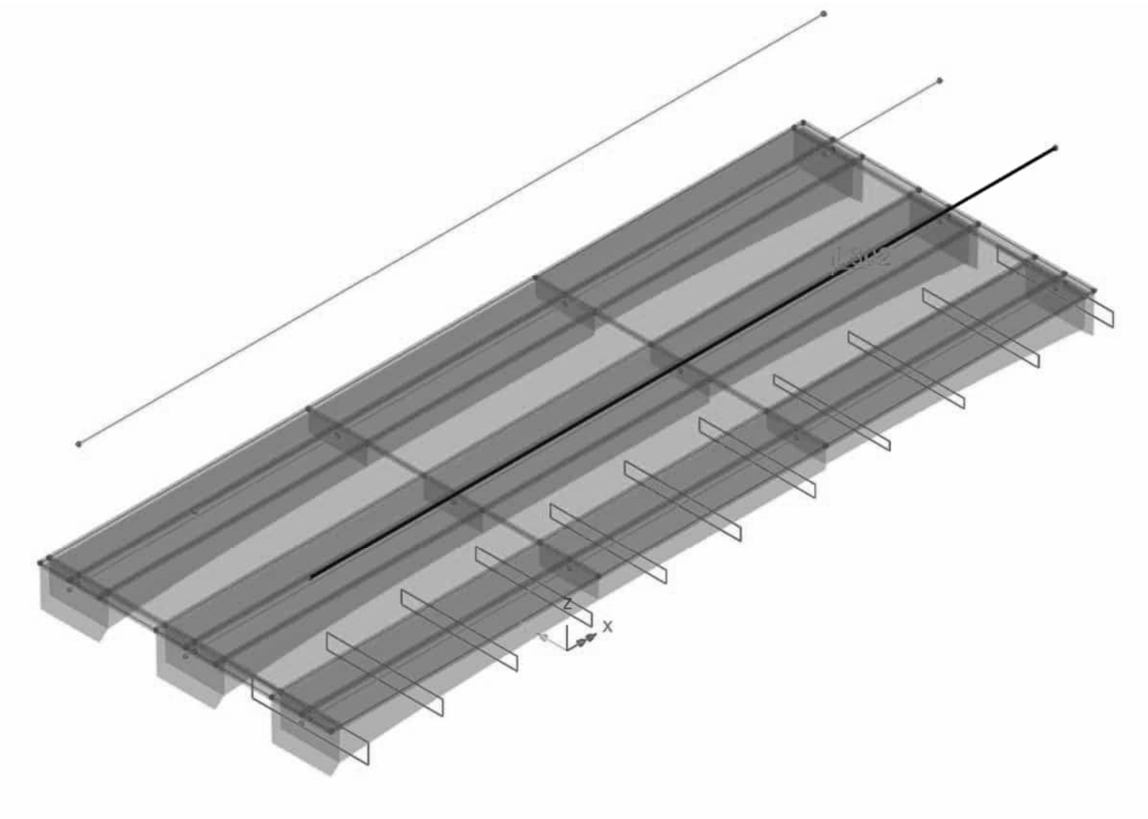




OVERVIEW

2.8.3 Slice beam LB3





OVERVIEW

2.9 FLANGE WIDTH

Flange width is determined by SS-EN 1992-1-1 section 5.3.2.1.

Verification shows that no reduction cross section is needed in “midspan”.

I are area 5 m (∴ 0.15 m ·33.6 m) from abutment a reduction of cross section is needed when studying capacity.

$l_1 = 8.2m$: length abutment

$l_2 = 33.6m$: length superstructure

Verification ”midspan” :

$l_0 = 0.70l_2 = 0.70 \cdot 33.6m = 23.5m$

$\min(0.2b_1 + 0.1l_0; 0.2l_0, b_1) = \min(0.2 \cdot 1.25m + 0.1 \cdot 23.5m\,; 0.2 \cdot 22.1m, 1.25m) = 1.25m$

$\rightarrow b_{ef.1} = 1.25m$

Verification “support” :

$l_0 = 0.15 \cdot (l_1 + l_2) = 0.15 \cdot (8.2m + 33.6m) = 6.3m$

$\min(0.2b_1 + 0.1l_0; 0.2l_0, b_1) = \min(0.2 \cdot 1.25m + 0.1 \cdot 6.3m\,; 0.2 \cdot 6.3m, 1.25m) = 0.88m$

$\rightarrow b_{ef.1} = 0.88m$

3. LOADS

3.1 DEAD WEIGHT

Load applied to Analysis : *Analysis 1*

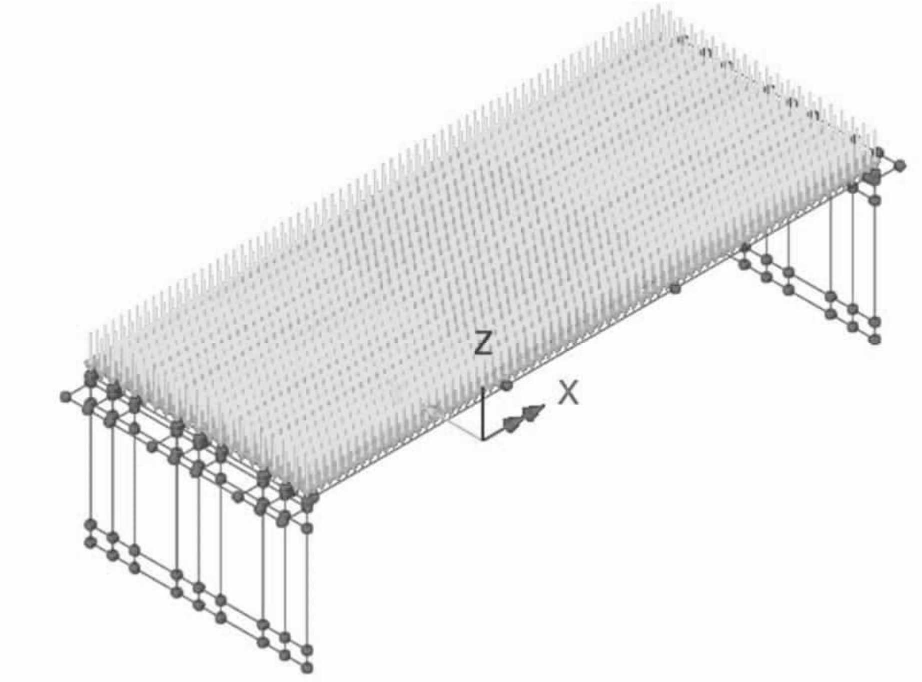
$\gamma_c = 25 \cdot \frac{kN}{m^3}$: concrete

3.1.1 Superstructure - deck

Load case : EGEN 1

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



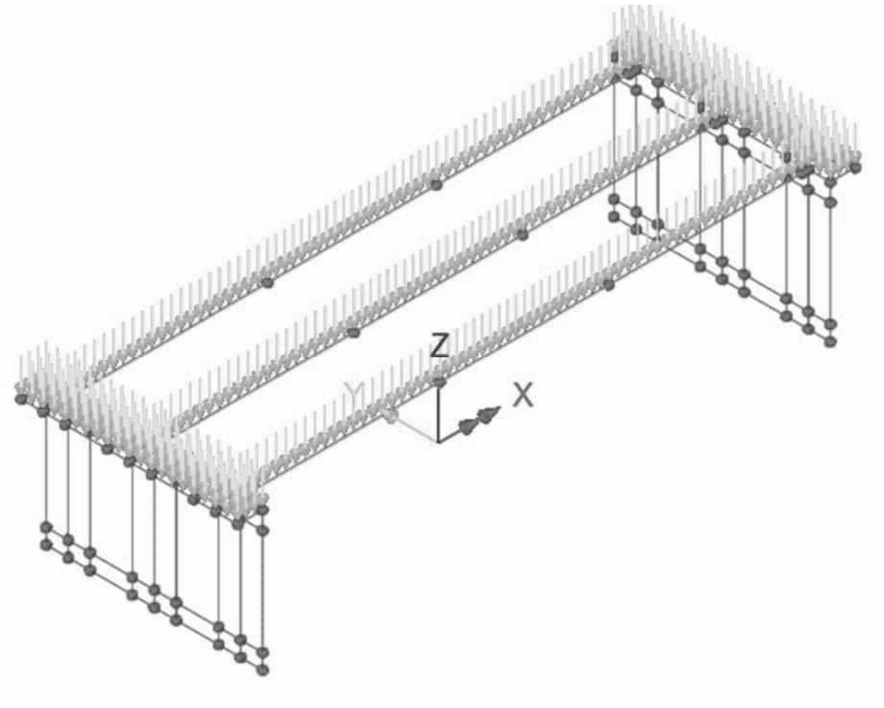
Overview 3D

3.1.2 Superstructure - beams

Loadcase : EGEN 2

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



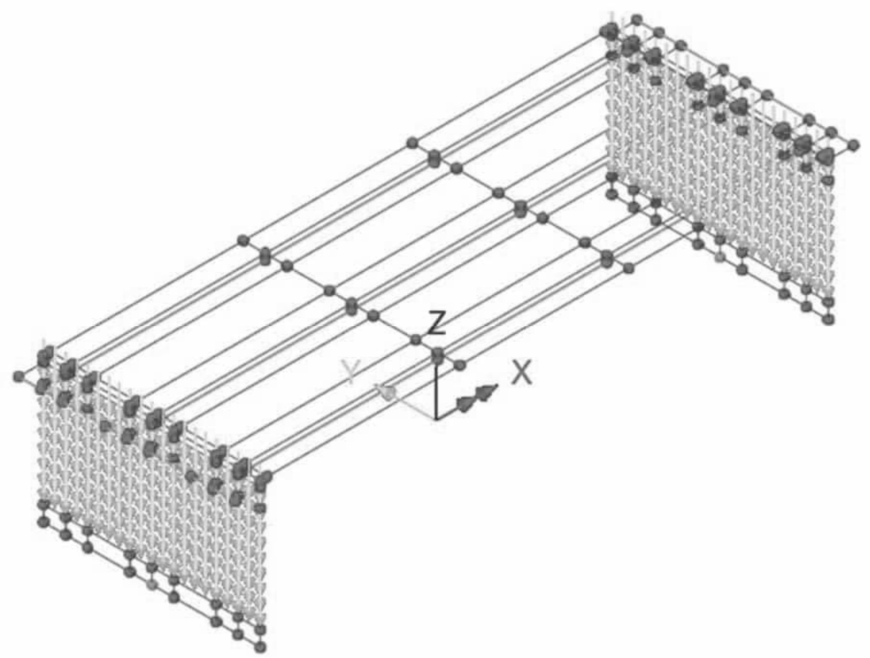
Overview 3D

3.1.3 Abutments

Loadcase : EGEN 3

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



Overview 3D

3.1.4 Edge beams including railing

Along each edge bream a line load is introduced. The load includes weight of edge beam and railing.

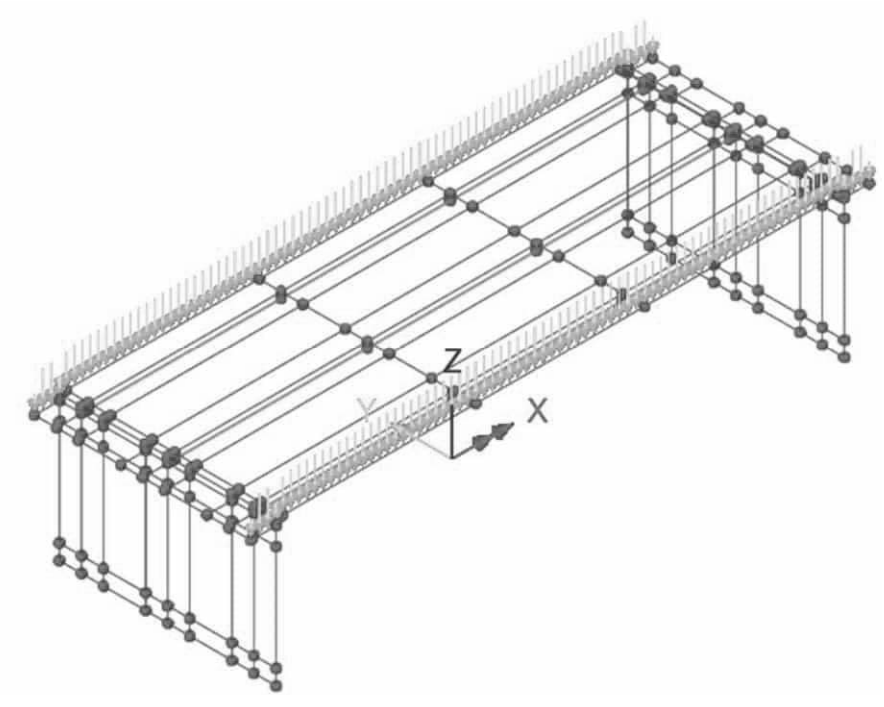
$p_{räcke} = 0.7 \frac{kN}{m}$: weight railing

$$\rightarrow p_z = p_{räcke} + p_{KB} = 0.7 \frac{kN}{m} + 0.40m \cdot 0.45m \cdot 25 \frac{kN}{m^3} = -6 \frac{kN}{m}$$

Loadcase : EGEN 4

Structural loading : Global distributed

Line load per unit length in Z direction: -6 $\frac{kN}{m}$



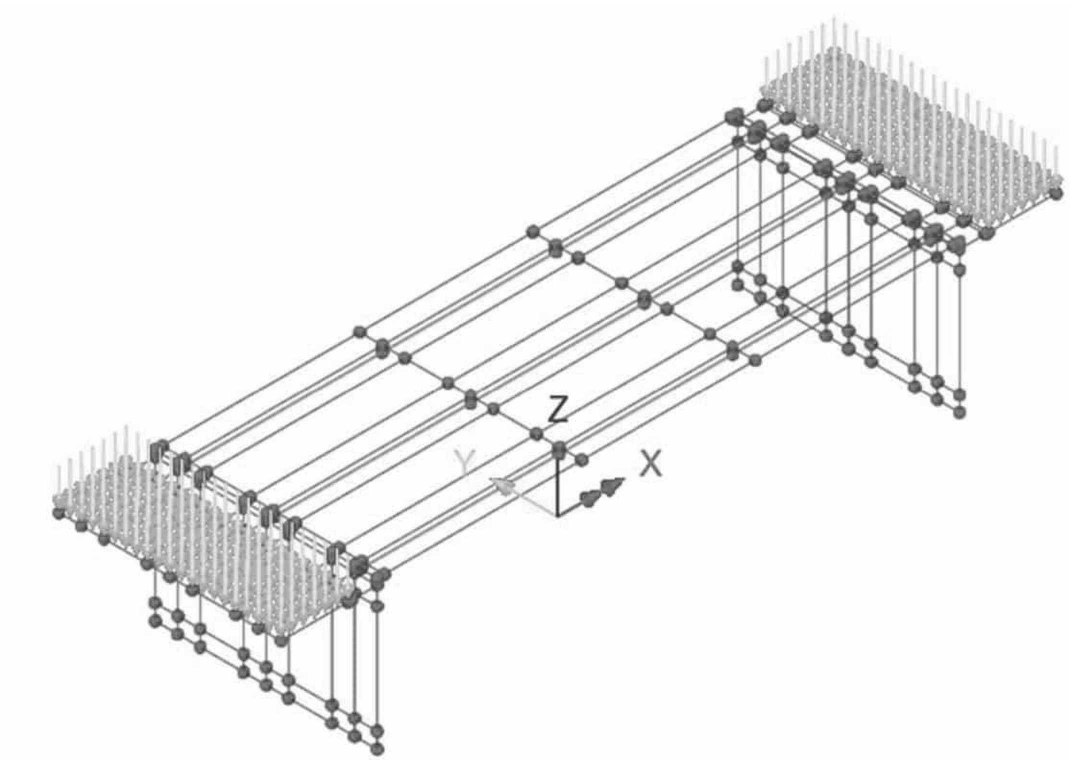
Overview 3D

3.1.5 Link slab

Loadcase : EGEN 5

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



Overview 3D

3.1.6 Wingwalls

All wingwalls are alike (∴ L = 6.0 m).

$P_z = -223kN$: page A3:30

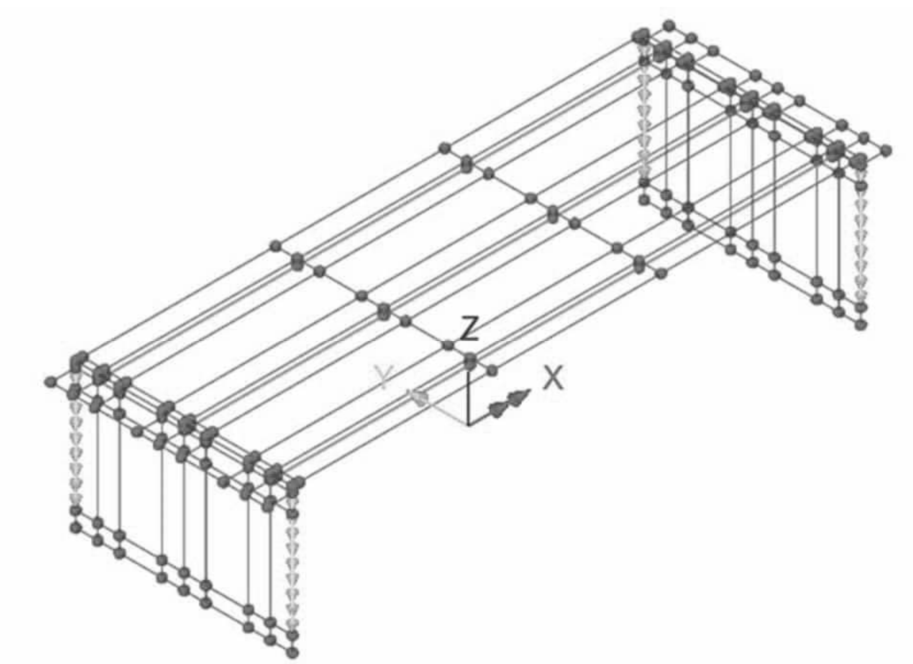
Load is distributed along edge of abutments from bottom of superstructure and distance 6.08 m downward.

$$p_z = \frac{P_z}{L} = -\frac{223N}{6.08m} = -37\frac{kN}{m}$$

Loadcase : EGEN 6

Structural loading : Global distributed

Line load per unit length in Z direction: -37 $\frac{kN}{m}$



Overview 3D

3.1.7 Load combination deadweight : EGEN

Basic load combination EGEN :

Loadcase	Factor
EGEN 1	1.00
EGEN 2	1.00
EGEN 3	1.00
EGEN 4	1.00
EGEN 5	1.00
EGEN 6	1.00

3.2 SURFACING

Load applied to Analysis : *Analysis 1*

3.2.1 Load on superstructure

Thickness pavement with a thickness of 95 mm built as follows:

- Wearing course ABS 11 — 40 mm
- Combined protective and binder course PGJA — 50 mm
- Waterproofing layer — 5 mm

$\gamma = 23\frac{kN}{m^3}$: course

$q_{matta} = 0.11kPa$: waterproofing

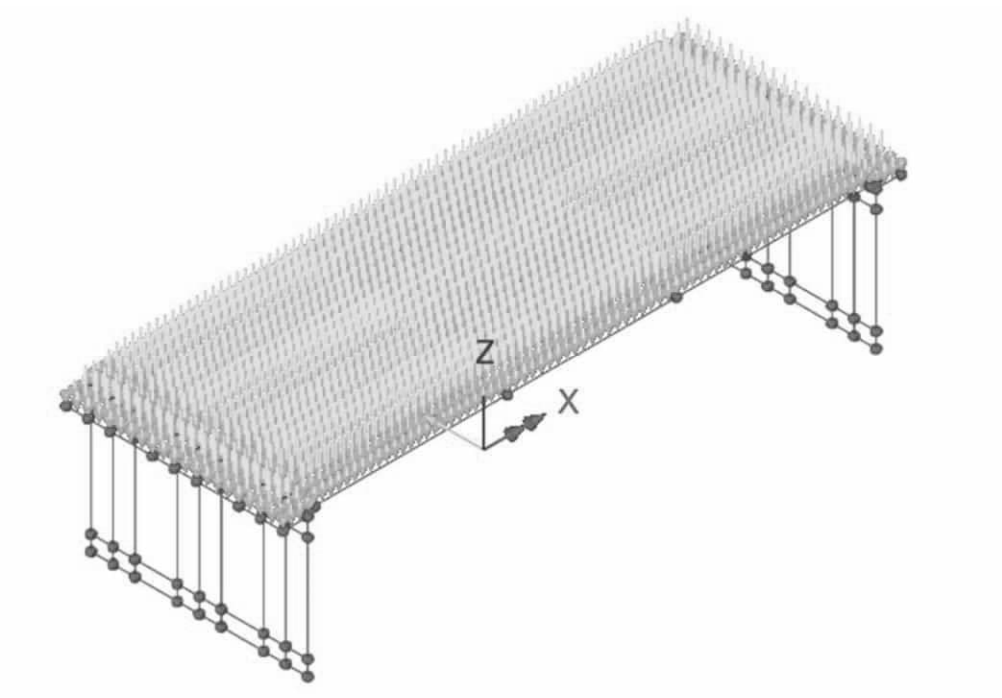
Surfacing load is seen below:

$q_{belägg} = \gamma_{beläggning} \cdot t + q_{matta} = 23\frac{kN}{m^3} \cdot 0.09m + 0.11kPa = 2.2kPa$

Load case : BELAGG 1

Structural loading : Global distributed

Surface load per unit area in Z direction: -3 $\frac{kN}{m^2}$



Overview 3D

3.2.2 Load on link slab

On the upper side of the link plate, there is a 95 mm pavement and an overfill with varying thickness (100-400 mm).

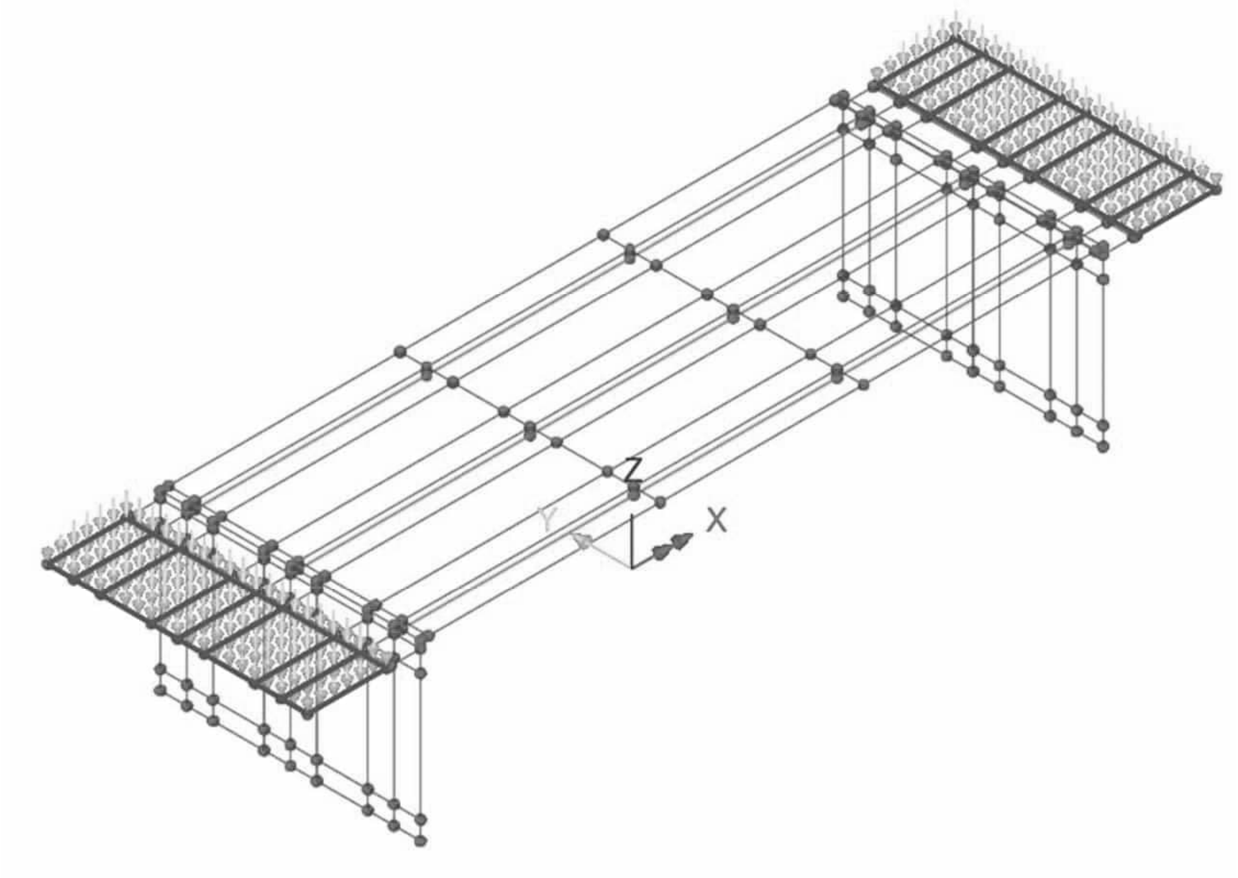
The overfill is considered equivalent to the base layer. In the static model, a fictitious load corresponding to the weights for the pavement and overfill is introduced (an average thickness of 250 mm is applied.

$$q_{belägg} = 2.2kPa + 22\frac{kN}{m^3} \cdot 0.25m = 8.0kPa$$

Load case : BELAGG 2

Structural loading : Global distributed

Surface load per unit area in Z direction: -8 $\frac{kN}{m^2}$



Overview 3D

3.2.3 Load combination surfacing: BELAGG

Basic load combination BELAGG :

Load case	Factor
BELAGG 1	1.00
BELAGG 2	1.00

3.3 EARTH PRESSURE

Load applied to Analysis : *Analysis 1*

Earth pressure in filling corresponds to coarse crushed blasted rock (AMA CEB.524).

φ_k = 45°

$\gamma = 20\frac{kN}{m^3}$

$\gamma' = 13\frac{kN}{m^3}$

$$X_d = \frac{1}{\gamma_m} \cdot \eta \cdot \overline{X} \equiv \frac{1}{\gamma_m} \cdot X_k$$

Earth pressure coefficient for design method 2 (D2) :
Design coefficients associated to *A1 + M1 + R2* according to SS-EN 1997-1 section 2.4.7.3.4.3 is applied.

$\gamma_{m.D2}$ = 1.0 : see TSFS chapter 38 table 38.3 for M1

$\rightarrow \varphi_d = artctan\left(\frac{tan\varphi_k}{\gamma_{m.D2}}\right) = arctan\left(\frac{tan45°}{1.0}\right) = 45°$

$K_0 = 1 - \sin(\varphi_d) = 0.29$

$K_a = tan^2\left(45° - \frac{\varphi_d}{2}\right) = 0.17$

$K_p = tan^2\left(45° + \frac{\varphi_d}{2}\right) = 5.82$

Earth pressure for design method 3 (D3) :
Design coefficients associated to *A1(design loads)* + *A2(geotechnical loads)* + *M2* + *R3* according to SS-EN 1997-1 section 2.4.7.3.4.4 is applied.

$\gamma_{m.D3}$ = 1.3 : see TSFS chapter 8 table 38.3 för M2

$\rightarrow \varphi_d = artctan\left(\frac{tan\varphi_k}{\gamma_m}\right) = arctan\left(\frac{tan45°}{1.3}\right) = 38°$

Remark
These are not used used in FEM-analysis. This is done by adjusting load coefficients.

Earth pressure in FEM-analysis:

During design earth press coefficients associated to method D2 will used applied, however the load coefficients are adjusted according verification, see pages A3:170.

$K_0 = 0.29$

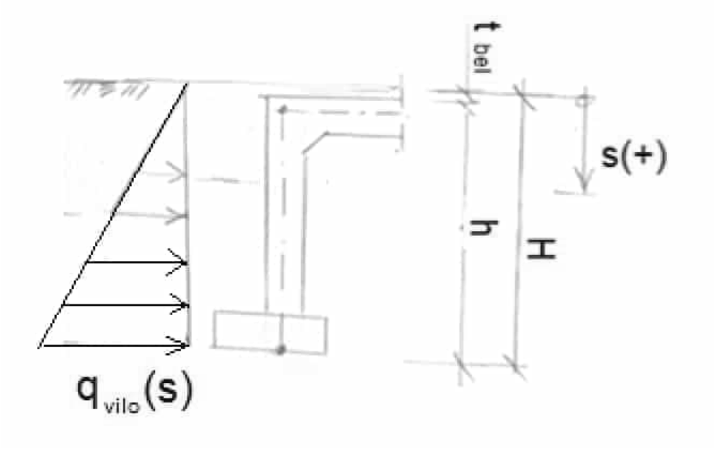
$K_a = 0.17$

$K_p = 5.82$

$h = 0.90m + 6.085m + 1.70m = 8.685m$

$H = 0.095m + 8.685m = 8.78m$

$q_{vilo}(s) = K_0 \cdot \gamma \cdot s = 0.29 \cdot 20\frac{kN}{m^3} \cdot s(+) = s(+) \cdot 5.8kPa$



Remark

Load width B = 12.8 is used for the bottom slab even though width B = 13.3 m. This simplification is possible since favorable effect of passive earth pressure has not been considered on safe side.

3.3.1 Load against abutment 1

$q_{vilo}\ (0m) = 0kPa$: top surfacing

$q_{vilo}\ (8.78m) = 8.78m \cdot 5.8\frac{kN}{m^3} = 51kPa$: underside bottom slab

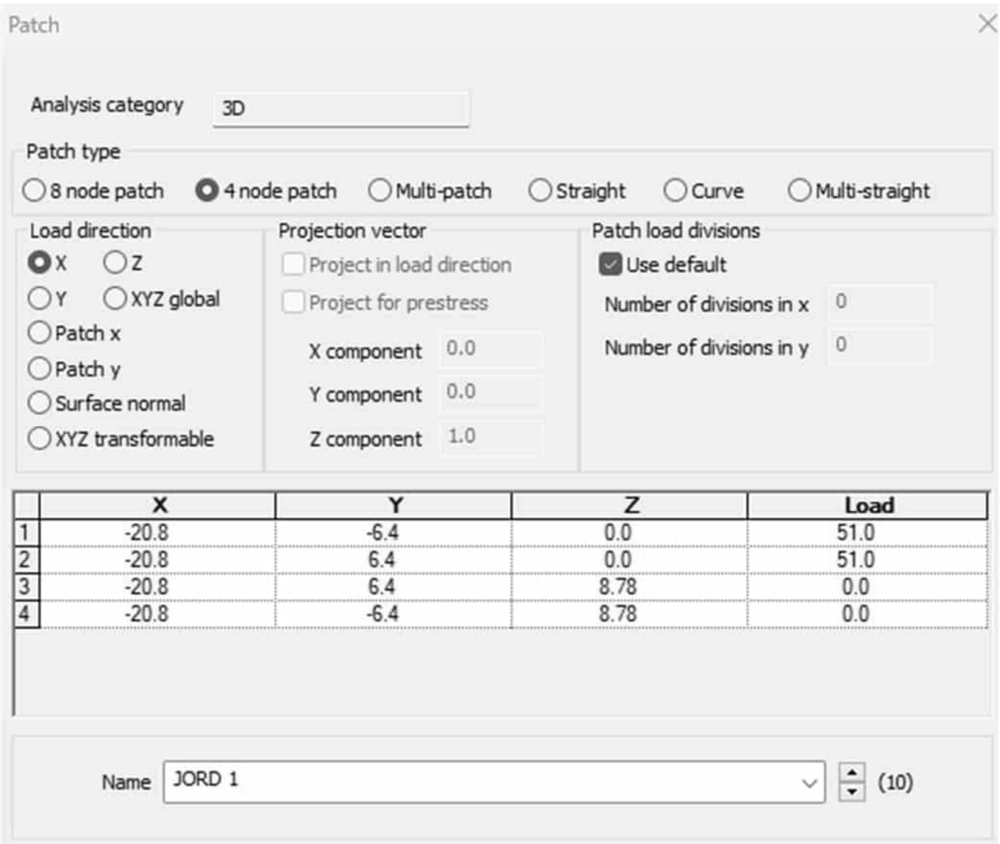
Loadcase : JORD 1

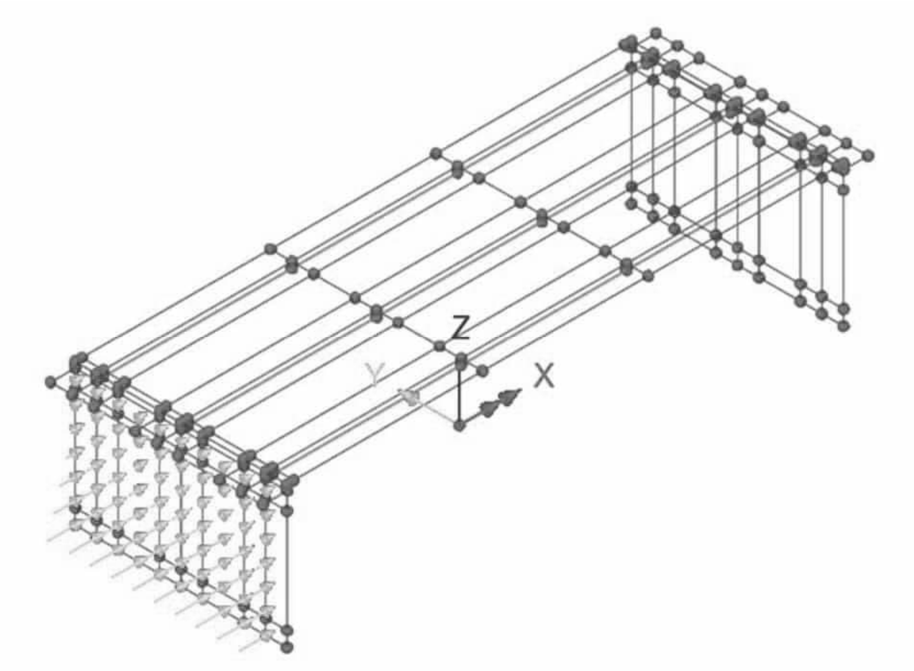
Structural loading : Discrete 4 node patch

Surface load (q_x) : 0 kPa → +51 kPa

Search Area : Abutment 1

Loads outside search area : Include full load





Overview 3D

3.3.2 Load against abutment 2

$q_{vilo}\ (0m) = 0kPa$: top surfacing

$q_{vilo}\ (8.78m) = 8.78m \cdot 5.8 \frac{kN}{m^3} = 51kPa$: underside bottom slab

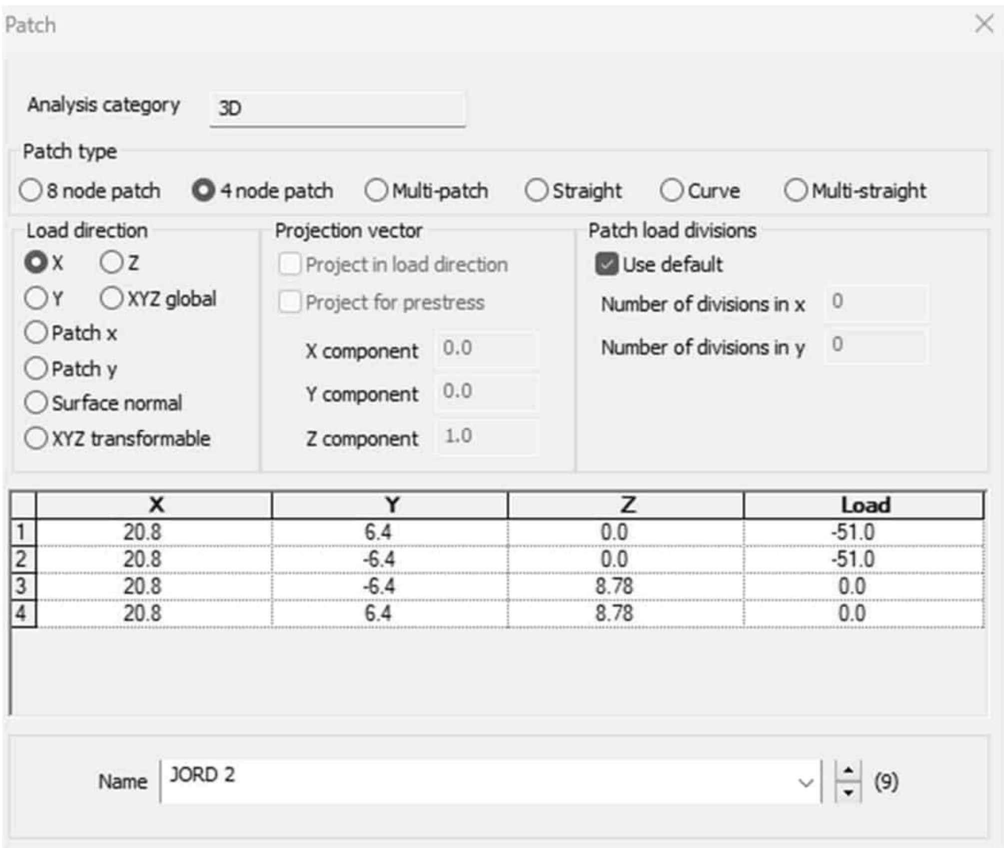
Loadcase : JORD 2

Structural loading : Discrete 4 node patch

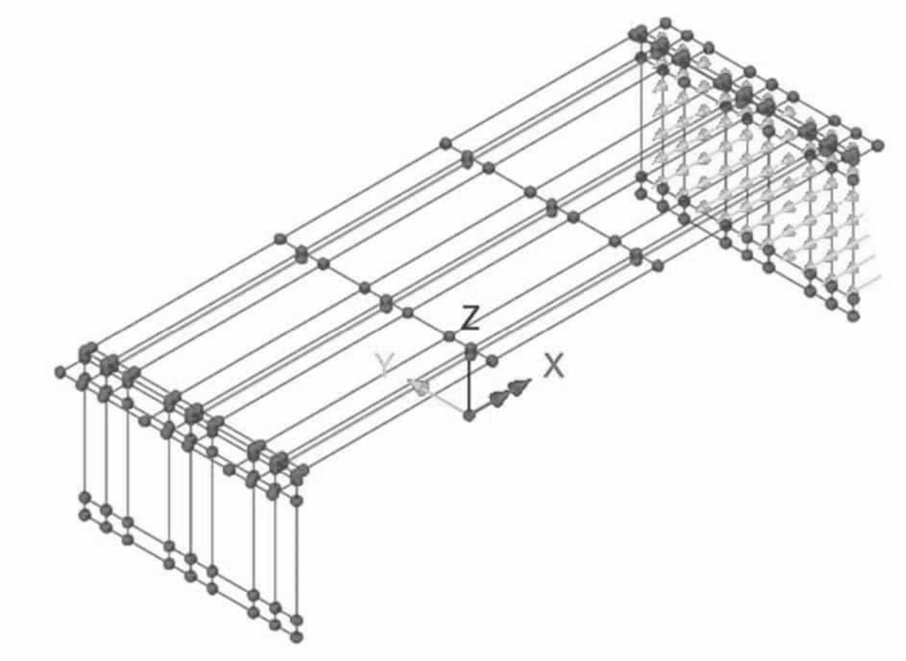
Surface load (q_x) : 0 kPa → -51 kPa

Search Area : Abutement 2

Loads outside search area : Include full load



	X	Y	Z	Load
1	20.8	6.4	0.0	-51.0
2	20.8	-6.4	0.0	-51.0
3	20.8	-6.4	8.78	0.0
4	20.8	6.4	8.78	0.0

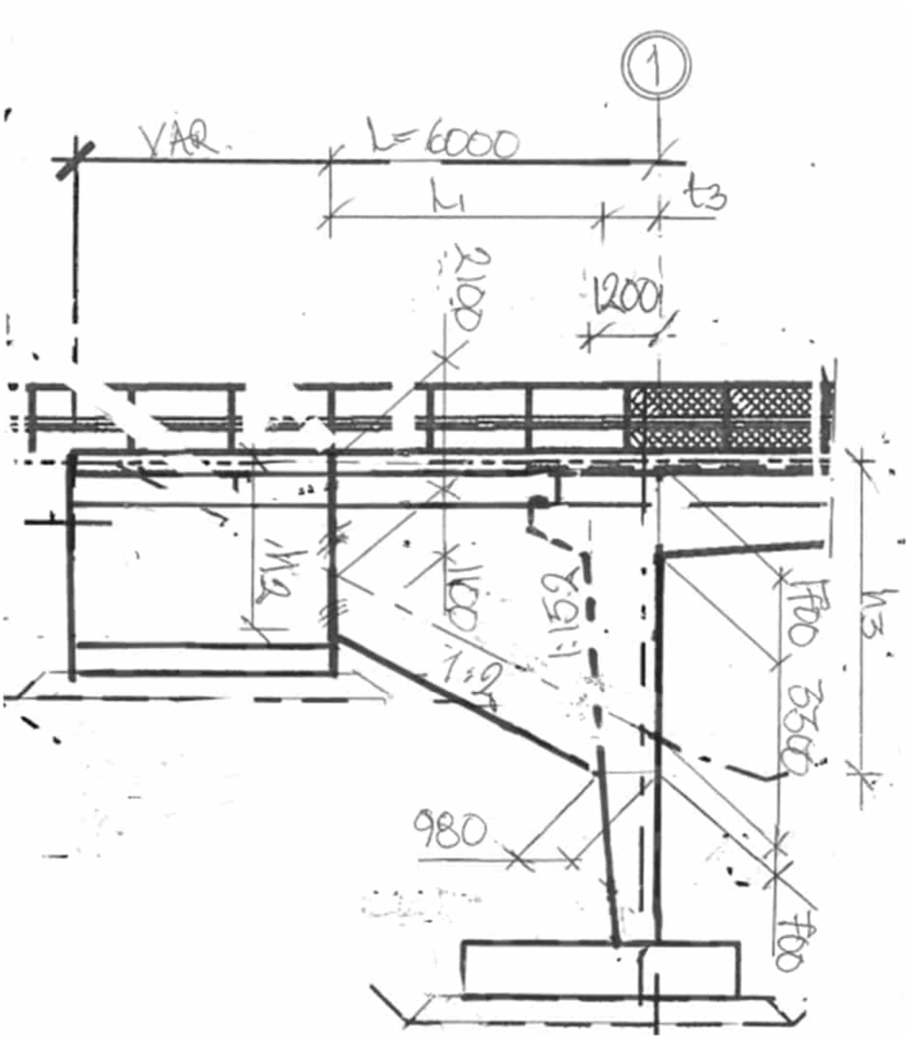


Overview 3D

3.3.3 Load against wingwalls

Design software K2.002 is used to determine earth pressure against wingwalls according to Culmans´s theory. All wingwalls are assumed to have same length (L = 6 m).

Load is distributed along edge of abutments from bottom of superstructure and distance 6.08 m downward. This assumption is on safe side.



$$t_3 = \frac{0.98m + 1.20m}{2} = 1.09m \rightarrow L_1 = L - t_3 = 6.0\ m - 1.09\ m = 4.91\ m$$

$$h_2 = 0.10m + 2.10m + 1.10m = 3.30m$$

$$h_3 = 0.10m + 1.70m + 4.00m = 5.80m$$

$$h_4 = 0.10m$$

Effective height along edge abutment:

$H_{ef} = 5.0m$: see page A3:30

Load at abutment edge quasi-load status (SLS-Q):

$N_{SLS-Q} = +81 \frac{kNm}{m}$: see page A3:30

$M_{SLS-Q} = 207 \frac{kNm}{m}$: see page A3:30

Characteristic earth pressure at abutment edge:

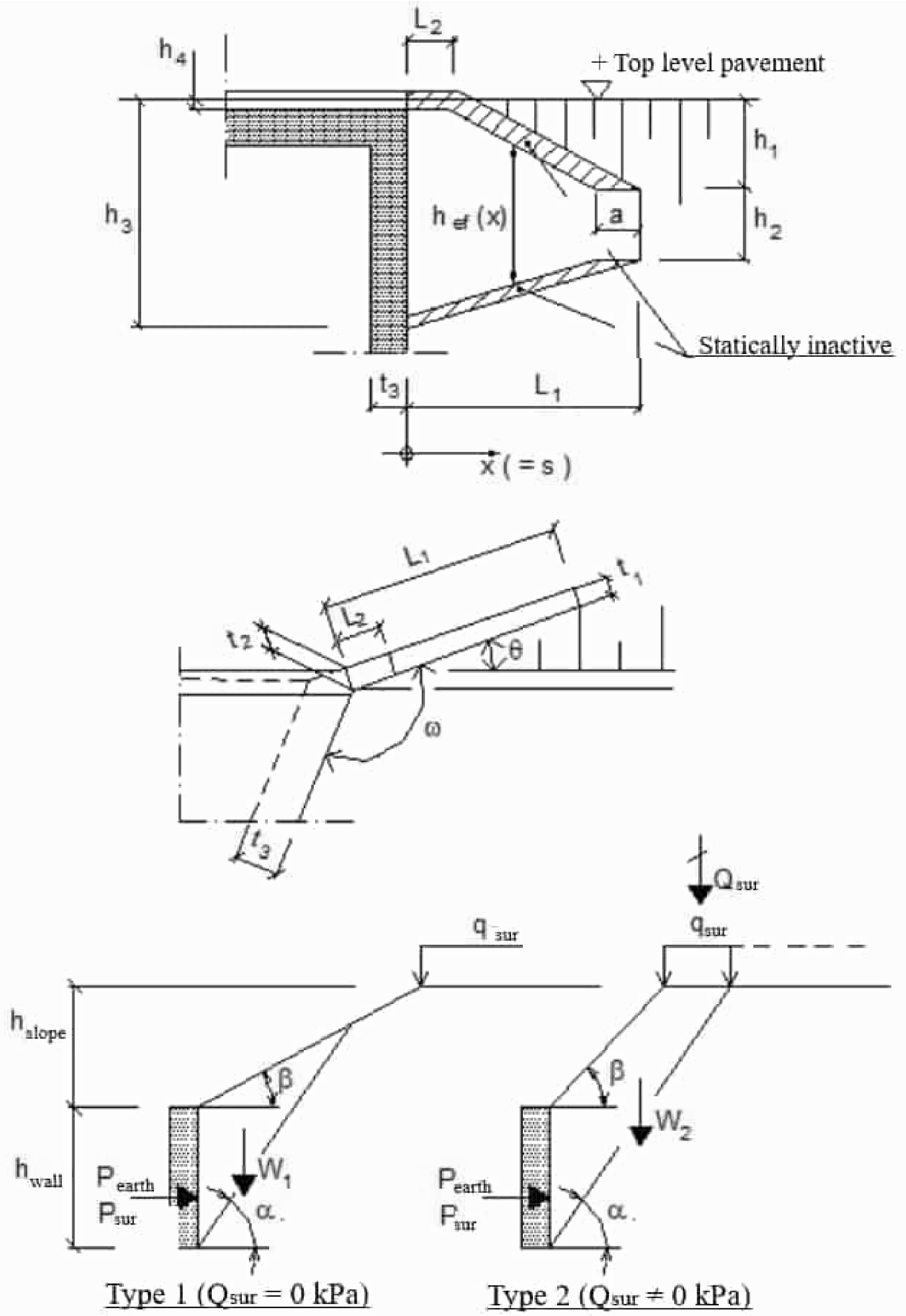
Load below is distributed along length 6.08 m in FEM-model.

$$N_{jord} = +81 \frac{kN}{m} \cdot \frac{1}{1.34} = 60 \frac{kN}{m}$$

$$M_{jord} = 207 \frac{kNm}{m} \cdot \frac{1}{1.34} = 154 \frac{kNm}{m}$$

Object : L = 6.0 m

PRINCIPLE SKETCH



Earth pressure method of Culman

INPUT

Geometry :

$L_1 := 4.91 \cdot m$ $L_2 := 1 \cdot mm$

$h_1 := 1 \cdot mm$ $h_2 := 3.30 \cdot m$ $h_3 := 5.80 \cdot m$ $h_4 := 0.10 \cdot m$

$t_1 := 0.40 \cdot m$ $t_2 := 0.40 \cdot m$ $t_3 := 1.09 \cdot m$

Angle wingwall-abutment: $\omega := 91 \cdot °$

Angle wingwall-road : $\theta := 1 \cdot °$

Distance from break point for effective height : $a := 1.4 \cdot m$

Material :

Soil material : $\gamma_{jord} := 20 \cdot \frac{kN}{m^3}$ $K_o := 0.29$ $K_a := 0.17$

Concrete : $\gamma_{btg} := 25 \cdot \frac{kN}{m^3}$

Loads :

Surcharge : $q_{över} := 20 \cdot kPa$

Load coefficients :

Earth pressure

$\psi\gamma_{ULS.1} := 1.49$ $\psi\gamma_{SLS.1} := 1.34$

Surcharge

$\psi\gamma_{ULS.2} := 1.71$ $\psi\gamma_{SLS.2} := 0$

CALCULATION

Earth pressure according method of Culman:

Nivå överkant vingmur : $mur_{ök} = linterp\left[(0m\ \ L_2\ \ L_1),(h_3\ \ h_3\ \ h_3 - h_1),s\right]$

Nivå underkant vingmur : $mur_{uk} = linterp\left[(0m\ \ L_1),(0m\ \ h_3 - h_1 - h_2),s\right]$

Vingmurens höjd : $h_{mur} = mur_{ök} - mur_{uk}$

Släntens höjd : $h_{slänt} = linterp\left[(0m\ \ L_2\ \ L_1),(0m\ \ 0m\ \ h_1),s\right]$

Friktionsvinkel: $\varphi = asin(1 - K_o)$

Lutning hos slänten ned till överkant vingmur mätt vinkelrätt mot vingen :

$$\beta = atan\left[\frac{h_1}{(L_1 - L_2)\tan(\theta)}\right]$$

Vertikallast för brottfigur typ 1 (= brottlinje i slänt, sålunda inget tillskott av överlast) :

$$W_1 = h_{mur} \cdot \sin\left(\frac{\pi}{2} - \alpha\right) \cdot \left(h_{mur} \cdot \cos\left(\frac{\pi}{2} - \alpha\right) + \frac{h_{mur} \cdot \sin\left(\frac{\pi}{2} - \alpha\right)}{\tan(\alpha - \beta)}\right) \cdot \frac{\gamma_{jord}}{2}$$

Vertikallaster för brottfigur typ 2 (= brottlinje hamnar ovanför slänt vilket ger ett bidrag från överlast) :

$$W_2 = \left[\left(h_{mur} + h_{slänt}\right)^2 \cdot \tan\left(\frac{\pi}{2} - \alpha\right) - \frac{h_{slänt}^2}{\tan(\beta)}\right] \cdot \frac{\gamma_{jord}}{2}$$

$$Q_{över}(q) = q \cdot \left[\left(h_{mur} + h_{slänt}\right) \cdot \tan\left(\frac{\pi}{2} - \alpha\right) - \frac{h_{slänt}(s)}{\tan(\beta)}\right]$$

Vilojordtrycksresultant enligt Culmann under inverkan av jordlast + överlast :

$$p_o(q) = \left| \begin{array}{l} W_{jord} \leftarrow W_1 \ \text{if} \ (h_{mur} + h_{slänt}) \cdot \tan(90° - \alpha) < \dfrac{h_{slänt}}{\tan(\beta)} \\ W_{jord} \leftarrow W_2 + Q_{över}(q) \ \ \text{otherwise} \\ p_{aktiv} \leftarrow W_{jord} \cdot \tan(\alpha - \varphi) \\ p_{aktiv} \cdot \dfrac{K_o}{K_a} \end{array} \right.$$

Utvärdera största last av jordtryck och överlast genom att kontrollera ett antal vinklar mellan φ och 90°. Överlastens lasteffekt fås som skillnaden mellan jordtrycksresultant med och utan överlast

$$p_{jord}(s) = \left| \begin{array}{l} N_\alpha \leftarrow 20st \\ \Delta\alpha \leftarrow \dfrac{90° - \varphi}{N_\alpha - 1} \\ \alpha \leftarrow \varphi \\ p_{max} \leftarrow p_o(0kPa) \\ \text{for } i \in 2..N_\alpha \\ \quad \left| \begin{array}{l} \alpha \leftarrow \alpha + \Delta\alpha \\ p_{vilo} \leftarrow p_o(0kPa) \\ \text{if } p_{vilo} > p_{max} \\ \quad \left| \begin{array}{l} p_{max} \leftarrow p_{vilo} \\ \alpha_{max} \leftarrow \alpha \end{array} \right. \end{array} \right. \end{array} \right.$$

$$p_{över}(s) = \left| \begin{array}{l} N_\alpha \leftarrow 20st \\ \Delta\alpha \leftarrow \dfrac{90° - \varphi}{N_\alpha - 1} \\ \alpha \leftarrow \varphi \\ p_{max} \leftarrow p_o(q_{över}) - p_o(0kPa) \\ \text{for } i \in 2..N_\alpha \\ \quad \left| \begin{array}{l} \alpha \leftarrow \alpha + \Delta\alpha \\ p_{över} \leftarrow p_o(q_{över}) - p_o(0kPa) \\ \text{if } p_{över} > p_{max} \\ \quad \left| \begin{array}{l} p_{max} \leftarrow p_{över} \\ \alpha_{max} \leftarrow \alpha \end{array} \right. \end{array} \right. \end{array} \right.$$

Forces earth pressure & surcharge :

$$H_{jord}(x_s) = \int_{x_s}^{L_1} p_{jord}(s)\, ds \qquad M_{jord}(x_s) = \int_{x_s}^{L_1} (s - x_s) \cdot p_{jord}(s)\, ds$$

$$H_{över}(x_s) = \int_{x_s}^{L_1} p_{över}(s)\, ds \qquad M_{över}(x_s) = \int_{x_s}^{L_1} (s - x_s) \cdot p_{över}(s)\, ds$$

Lastkombinering - Lk ULS och Lk SLS :

Snittkraft i frontmur för inspänningssnitt :

$$N'_{ULS.front} = \left(\psi\gamma_{ULS.1} \cdot H'_{jord}(0 \cdot m) + \psi\gamma_{ULS.2} \cdot H'_{över}(0 \cdot m)\right) \cdot \sin(\alpha)$$

$$M'_{ULS.front} = \psi\gamma_{ULS.1} \cdot M'_{jord}(0 \cdot m) + \psi\gamma_{ULS.2} \cdot M'_{över}(0 \cdot m) + N'_{ULS.front} \cdot \frac{t_3}{2}$$

$$N'_{SLS.front} = \left(\psi\gamma_{SLS.1} \cdot H'_{jord}(0 \cdot m) + \psi\gamma_{SLS.2} \cdot H'_{över}(0 \cdot m)\right) \cdot \sin(\alpha)$$

$$M'_{SLS.front} = \psi\gamma_{SLS.1} \cdot M'_{jord}(0 \cdot m) + \psi\gamma_{SLS.2} \cdot M'_{över}(0 \cdot m) + N'_{SLS.front} \cdot \frac{t_3}{2}$$

Snittkrafter i vingmur :

$$Q'_{ULS}(s) = \psi\gamma_{ULS.1} \cdot H'_{jord}(s) + \psi\gamma_{ULS.2} \cdot H'_{över}(s)$$

$$M'_{ULS}(s) = \psi\gamma_{ULS.1} \cdot M'_{jord}(s) + \psi\gamma_{ULS.2} \cdot M'_{över}(s)$$

$$M'_{SLS}(s) = \psi\gamma_{SLS.1} \cdot M'_{jord}(s) + \psi\gamma_{SLS.2} \cdot M'_{över}(s)$$

Beräkning av effektiv höjd :

$$\Delta h = h_3 - h_2 - h_1$$ $\Delta h = 2.499\ m$

Nivå överkant effektiv vingmur :

$$Nivå_{ök} = \text{linterp}\left[(0m \;\; L_2 \;\; L_1 - a \;\; L_1), (h_3 - h_1 \;\; h_3 - h_4 \;\; h_3 - h_1 \;\; h_3 - h_1), s\right]$$

Nivå underkant effektiv vingmur :

$$Nivå_{uk} = \text{linterp}\left[(0m \;\; L_1 - L_2 \;\; L_1), \left(\frac{a}{L} \cdot \Delta h \;\; \Delta h \;\; \Delta h\right), s\right]$$

Effektiv höjd vingmur :

$$h_{ef}(s) = Nivå_{ök} - Nivå_{uk}$$

Design forces (Lc ULS & Lc SLS) distributes over effective height :

Snittkraft i frontmur för inspänningssnitt :

$H_{ef} = h_{ef}(0m)$

$$N_{ULS.front} = \frac{N'_{ULS.front}}{H_{ef}}$$

$$M_{ULS.front} = \frac{M'_{ULS.front}}{H_{ef}}$$

$$N_{SLS.front} = \frac{N'_{SLS.front}}{H_{ef}}$$

$$M_{SLS.front} = \frac{M'_{SLS.front}}{H_{ef}}$$

Snittkrafter i vingmur :

$$Q_{ULS.ving} = \frac{Q'_{ULS}(s)}{h_{ef}(s)}$$

$$M_{ULS.ving} = \frac{M'_{ULS}(s)}{h_{ef}(s)}$$

$$M_{SLS.ving} = \frac{M'_{SLS}(s)}{h_{ef}(s)}$$

Dead weight wingwall :

$$t = t_2 - \frac{t_2 - t_1}{L_1} \cdot s$$

$A(s) = h_{mur} \cdot t$

$$V_{egen} = \gamma_{btg} \int_0^{L_1} A(s)\, ds$$

$$M_{egen} = \gamma_{btg} \int_0^{L_1} A(s) \cdot s\, ds$$

RESULTS

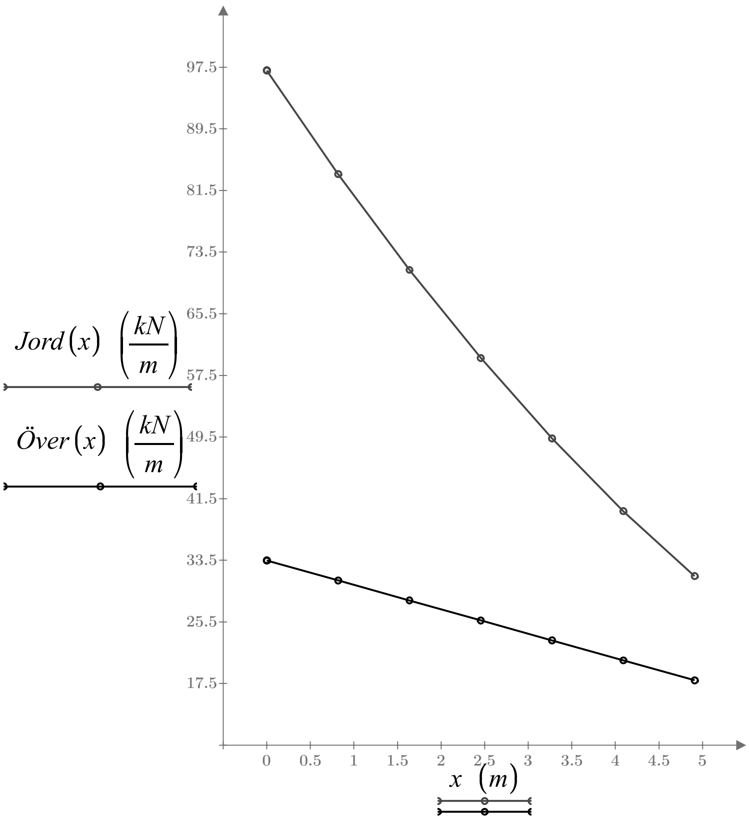
Partial results

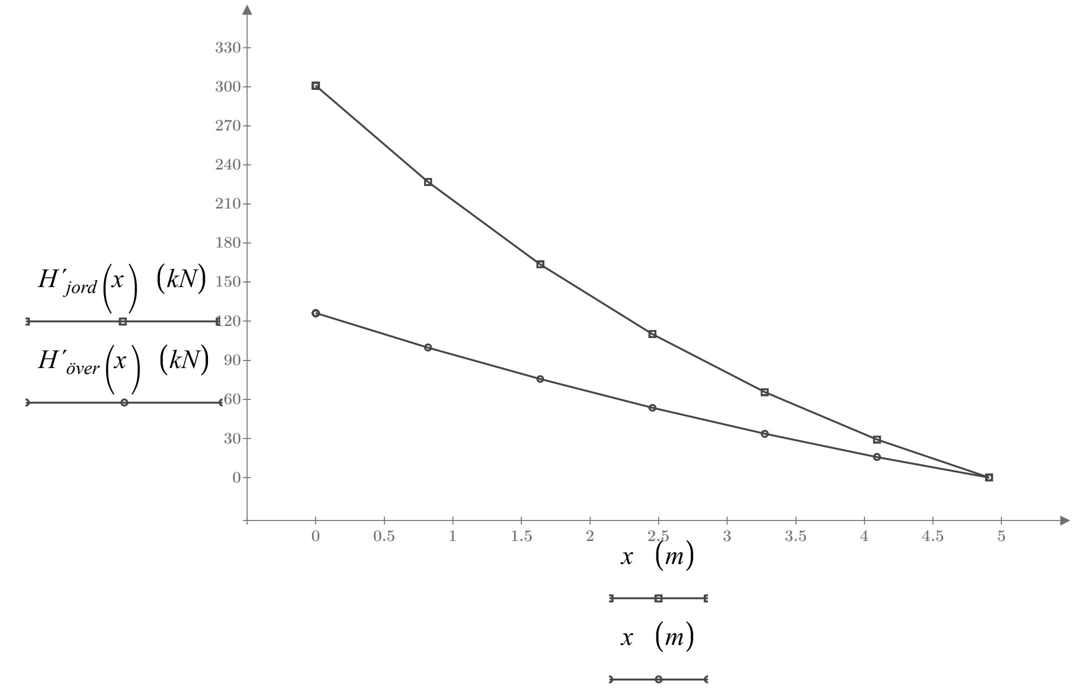
$\varphi = 45\ °$: design friction angle associated to K_o

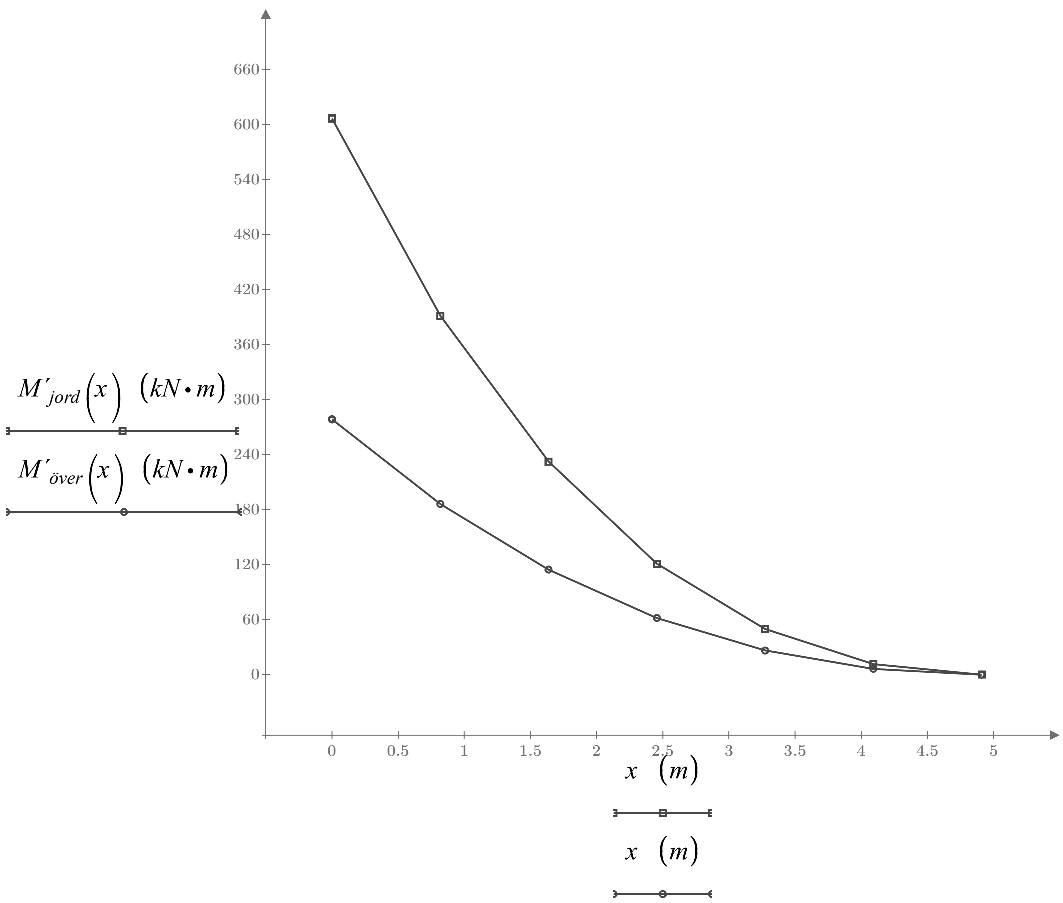
$\beta = 1\ °$: angle from top of road and to top of wing wall measured perpendicular to wingwall

Evaluation earth pressure, surcharge & associated critical rupture angle in table format :

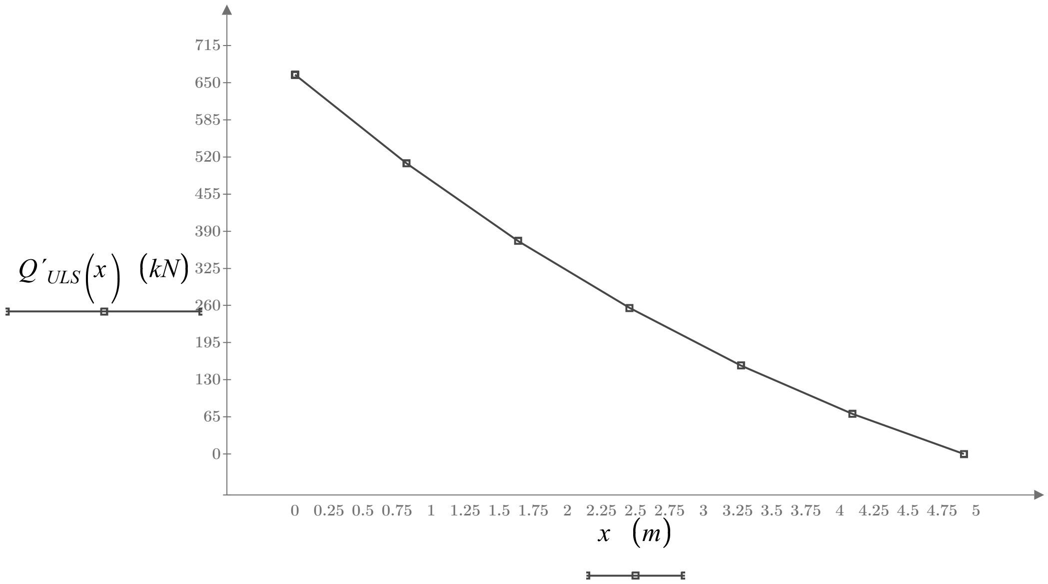
x	p_{jord}	α_{tillh}	$p_{över}$	α_{tillh}
0	97,1	66	33,5	66
0,000	97,1	66	33,5	66
0,001	97,1	66	33,5	66
0,001	97,1	66	33,5	66
0,82	83,6	66	30,9	66
1,64	71,2	66	28,3	66
2,46	59,8	66	25,7	66
3,27	49,3	66	23,1	66
4,09	39,9	66	20,5	66
4,91	31,4	66	17,9	66
m	kN/m	grader	kN/m	grader

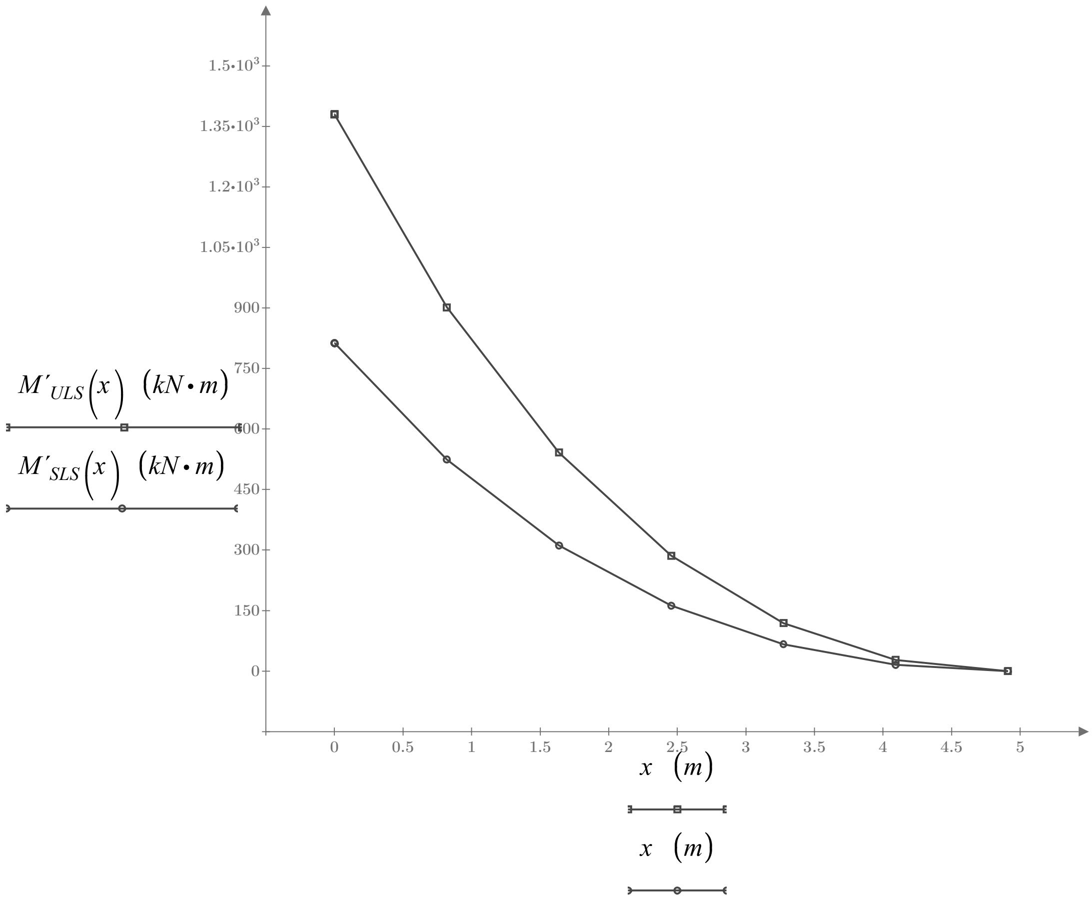



330
300
270
240
210
180
150
120
90
60
30
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
H´jord(x) (kN)
H´över(x) (kN)
x (m)
x (m)

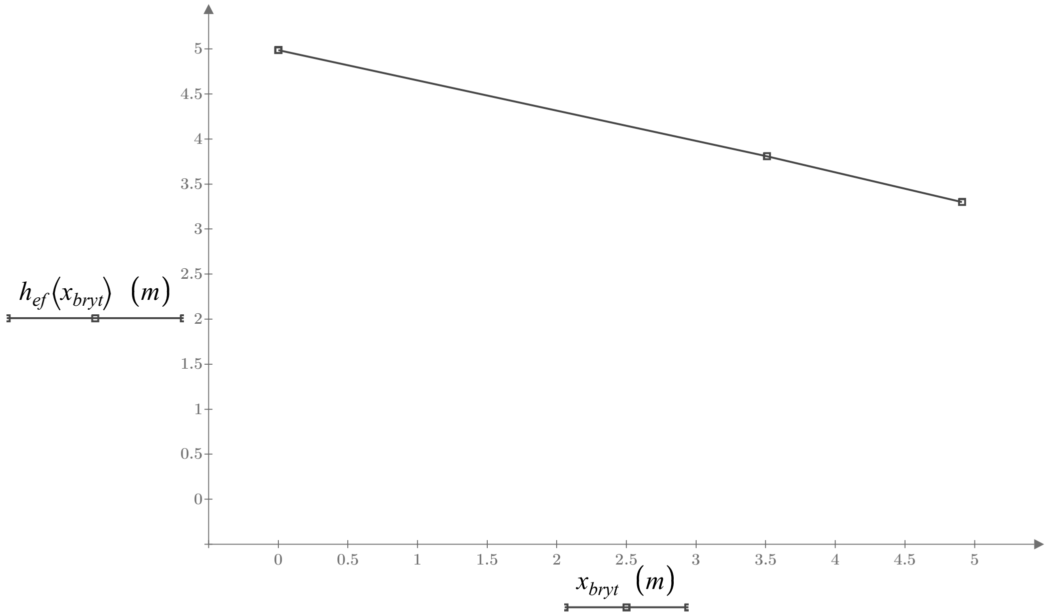

660
600
540
480
420
360
300
240
180
120
60
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
M´jord(x) (kN•m)
M´över(x) (kN•m)
x (m)
x (m)

Evaluation design forces associated to LC ULS & LC SLS :





Evaluation of effective height :



Detailed results

Design forces in section where wingwall is clamped to abutment:

Effective height in clamped section: $H_{ef} = 5.0\ m$

$N_{ULS.front}$	$M_{ULS.front}$	$N_{SLS.front}$	$M_{SLS.front}$
133	349	81	207
kN/m	kNm/m	kN/m	kNm/m

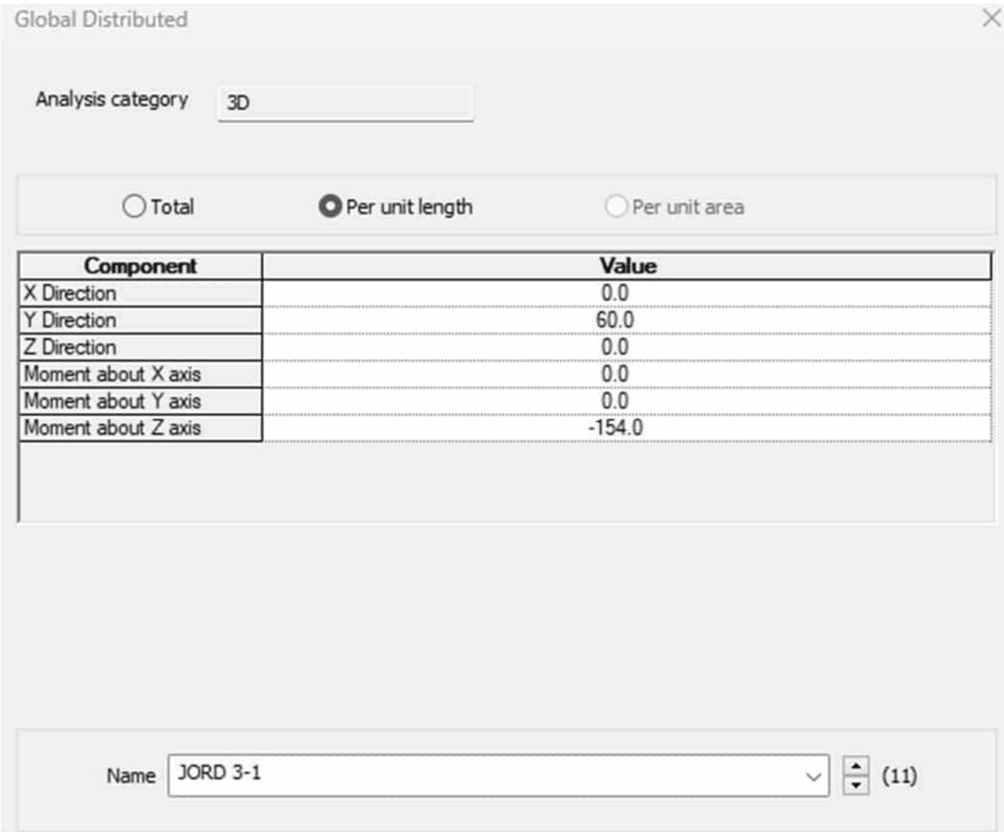
Design forces in wingwall:

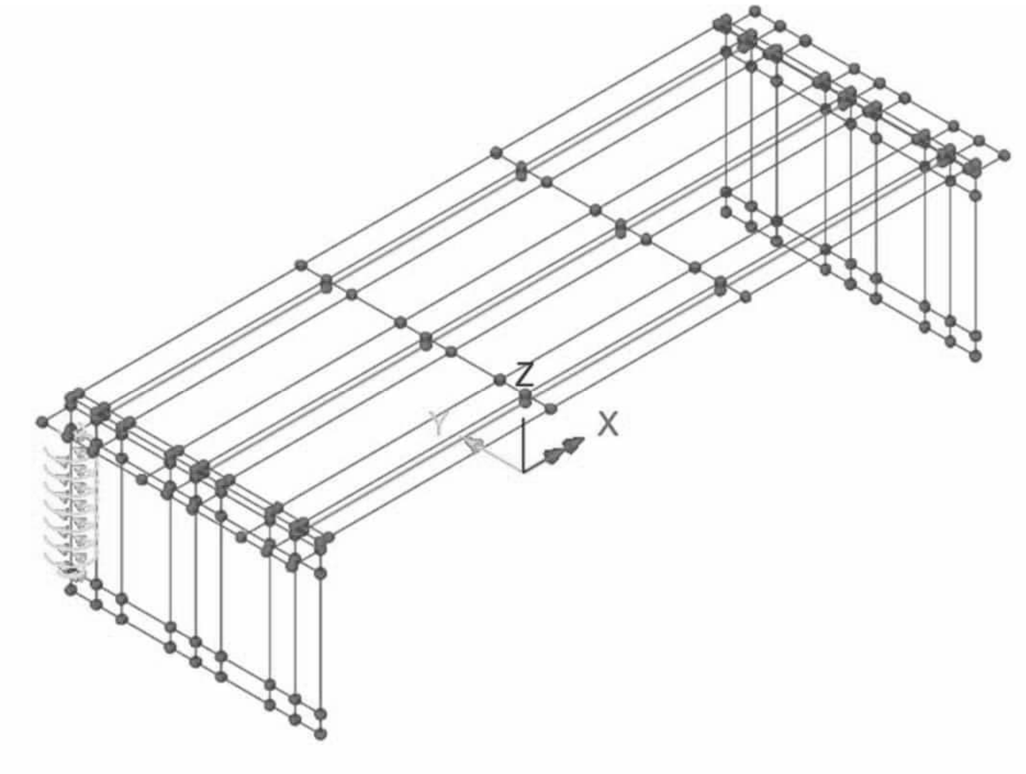
x	$Q_{ULS.ving}$	$M_{ULS.ving}$	$M_{SLS.ving}$	t (x)
0	133	277	163	0,400
0,000	133	277	163	0,400
0,001	133	277	163	0,400
0,001	133	277	163	0,400
0,82	108	191	111	0,400
1,64	84	122	70	0,400
2,46	61	69	39	0,400
3,27	40	31	17	0,400
4,09	20	8	4	0,400
4,91	0	0	0	0,400
m	kN/m	kNm/m	kNm/m	m

Load case: JORD 3-1
(Northern wingwall abutment 1)

$p_y = +60 \frac{kN}{m}$

$m_z = -154 \frac{kNm}{m}$



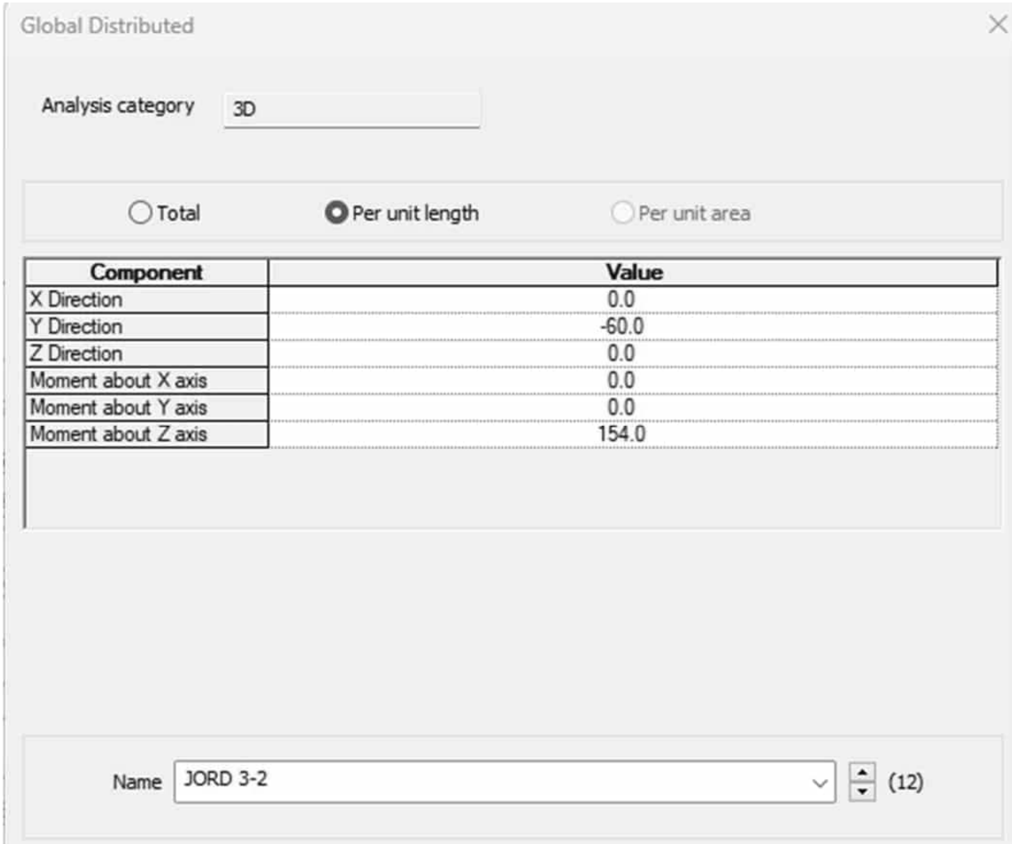


<u>Overview 3D</u>

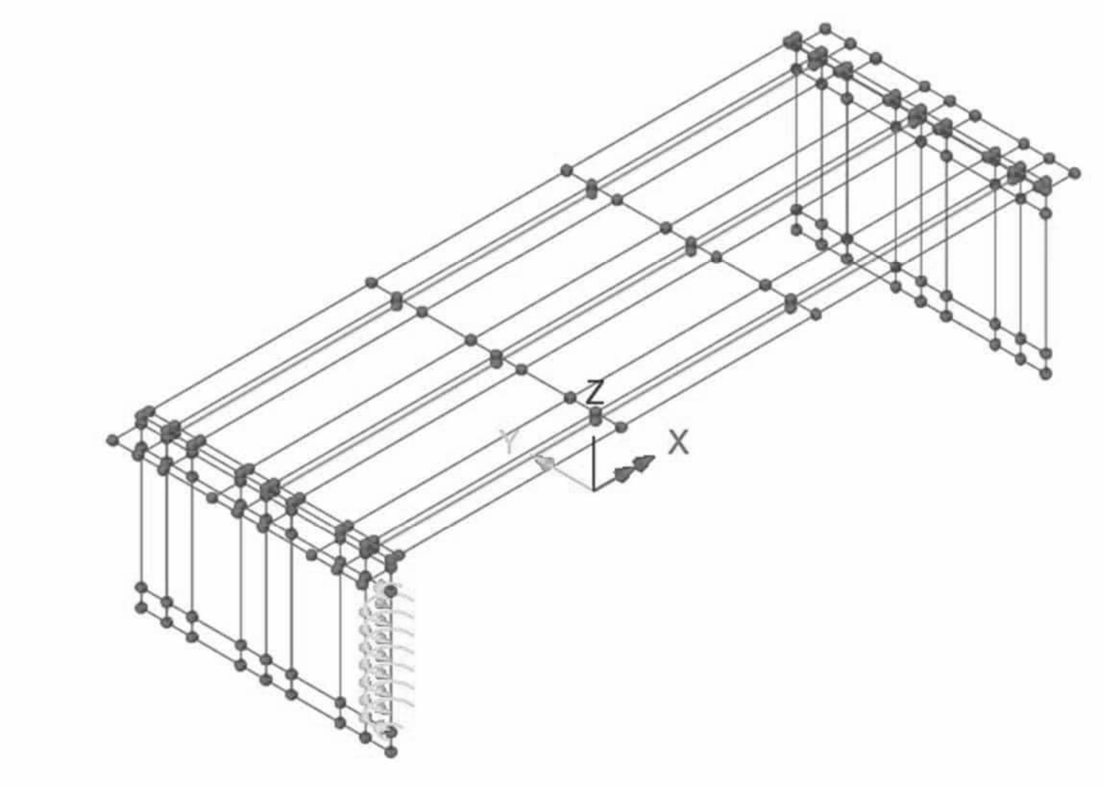
Load case : JORD 3-2
(Southern wingwall abutment 1)

$$p_y = -60 \frac{kN}{m}$$

$$m_z = +154 \frac{kNm}{m}$$



Component	Value
X Direction	0.0
Y Direction	-60.0
Z Direction	0.0
Moment about X axis	0.0
Moment about Y axis	0.0
Moment about Z axis	154.0

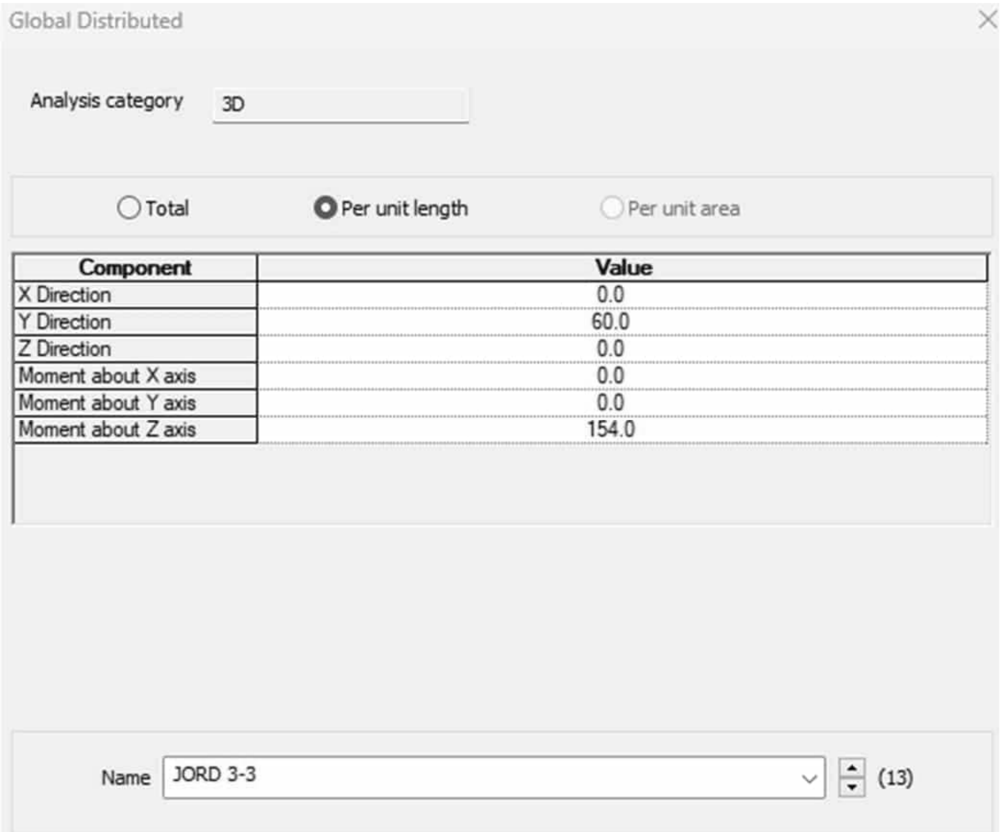


Overview 3D

Load case : JORD 3-3
(Northern wingwall abutment 2)

$$p_y = +60 \frac{kN}{m}$$

$$m_z = +154 \frac{kNm}{m}$$

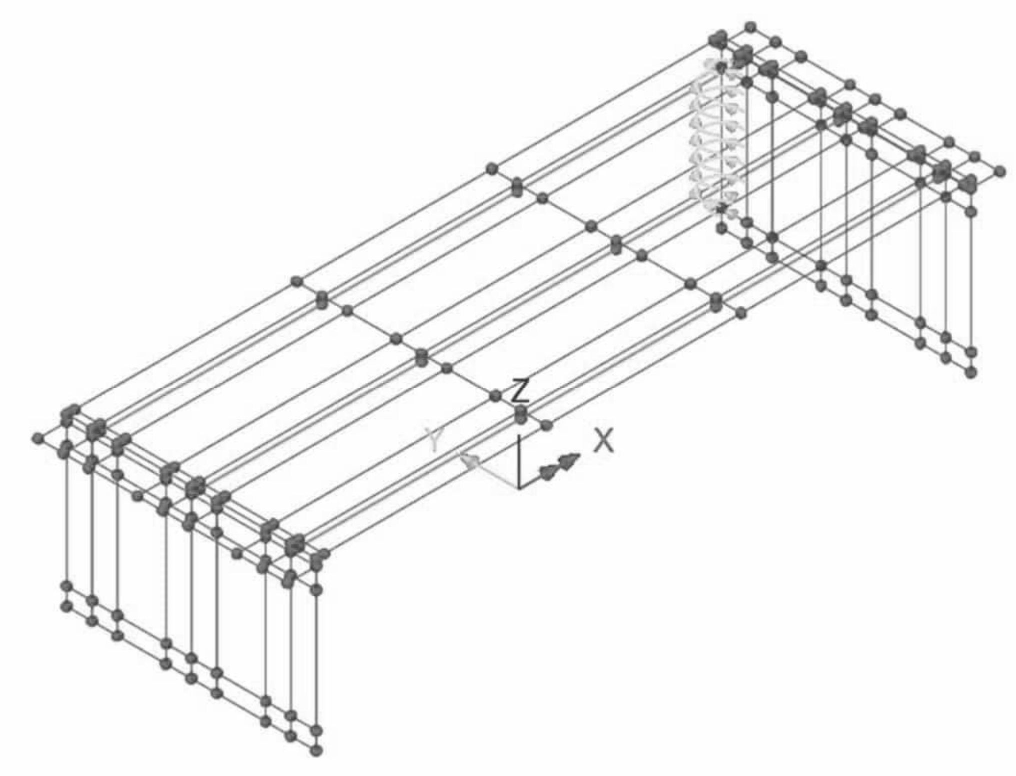

Global Distributed

Analysis category: 3D

Total / Per unit length (selected) / Per unit area

Component	Value
X Direction	0.0
Y Direction	60.0
Z Direction	0.0
Moment about X axis	0.0
Moment about Y axis	0.0
Moment about Z axis	154.0

Name: JORD 3-3 (13)

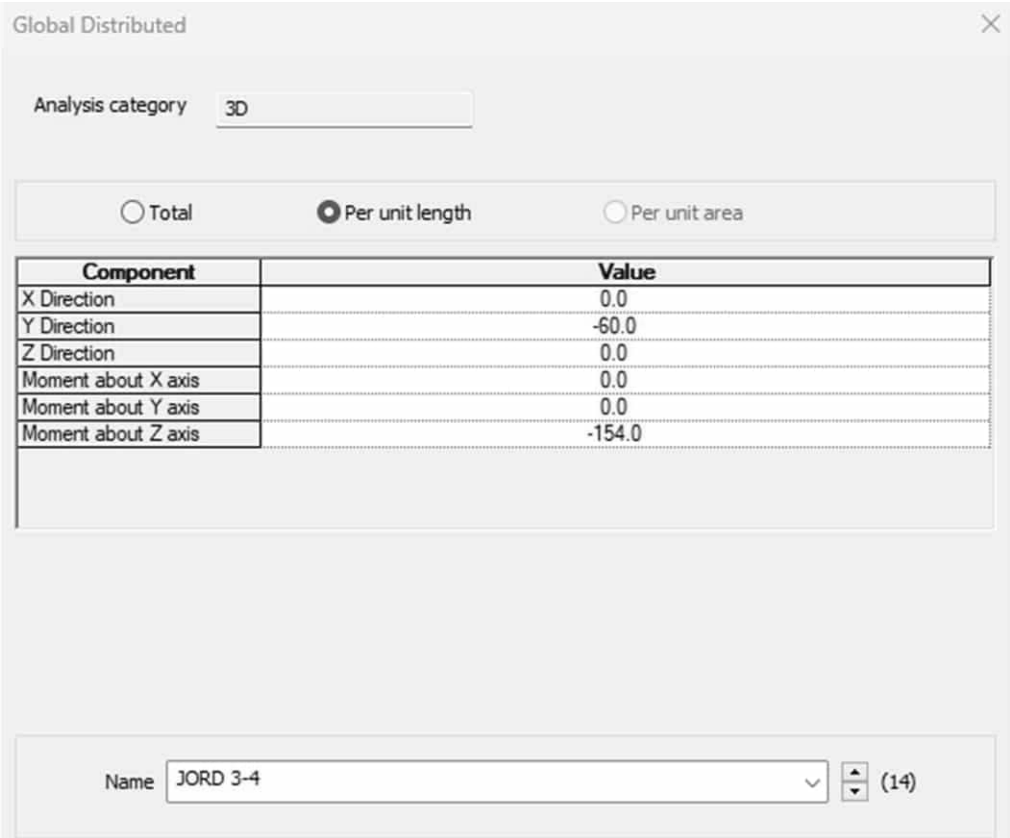


Overview 3D

Load case : JORD 3-4
(Southern wingwall abutment 2)

$$p_y = +60 \frac{kN}{m}$$

$$m_z = +154 \frac{kNm}{m}$$

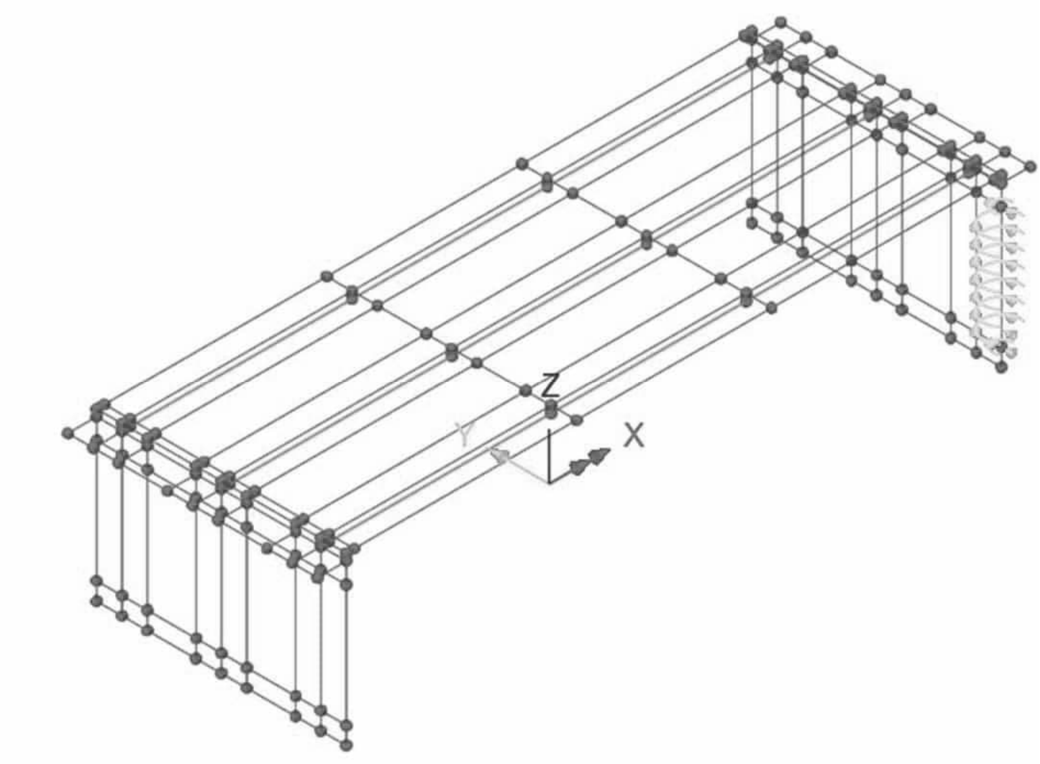

Global Distributed

Analysis category: 3D

Total / Per unit length (selected) / Per unit area

Component	Value
X Direction	0.0
Y Direction	-60.0
Z Direction	0.0
Moment about X axis	0.0
Moment about Y axis	0.0
Moment about Z axis	-154.0

Name: JORD 3-4 (14)



Overview 3D

3.3.4 Summary earth pressure: JORD

Combination of occurring load cases.

Basic loadcombination JORD :

Loadcase	Factor
JORD 1	1
JORD 2	1
JORD 3-1	1
JORD 3-2	1
JORD 3-3	1
JORD 3-4	1

3.4 SUPPORT SETTLEMENT

Load applied to Analysis : *Analysis 1*

Load effect of support settlement shall be considered in TRVINFRA-00227 section 7.2.1.1.1.1.

Only horizontal support displacement in the longitudinal direction of the bridge needs to be considered. Additionally, it is stated that horizontal and vertical support displacements do not need to be combined.

A horizontal support displacement in the longitudinal direction of the bridge (x-direction) of ±5 mm for each support is applied.

Vertical settlement difference (Z-direction) corresponding to support settlement of 10 mm is assumed to occur for all supports.

A verification will be performed to demonstrate that this is on the safe side.

Horizontal displacement (X-direction) amounts to ±10 mm.

When determining associated load effects, reduction is carried out with consideration to creep and cracking.

Note:
The impact of support settlement in serviceability limit state (SLS) according to SS-EN 1992-1-1 §2.3.1.3. If this occurs, a gradual crack development should be applied according to SS-EN 1992-1-1 §5.4(3). Reduction is carried out with consideration to creep and cracking.

The impact of support settlement is not considered in the ultimate limit state (ULS) according to SS-EN 1992-1-1 §2.3.1.3 for this type of bridge.

3.4.1 Vertical settlement

3.4.1.1 Support 1

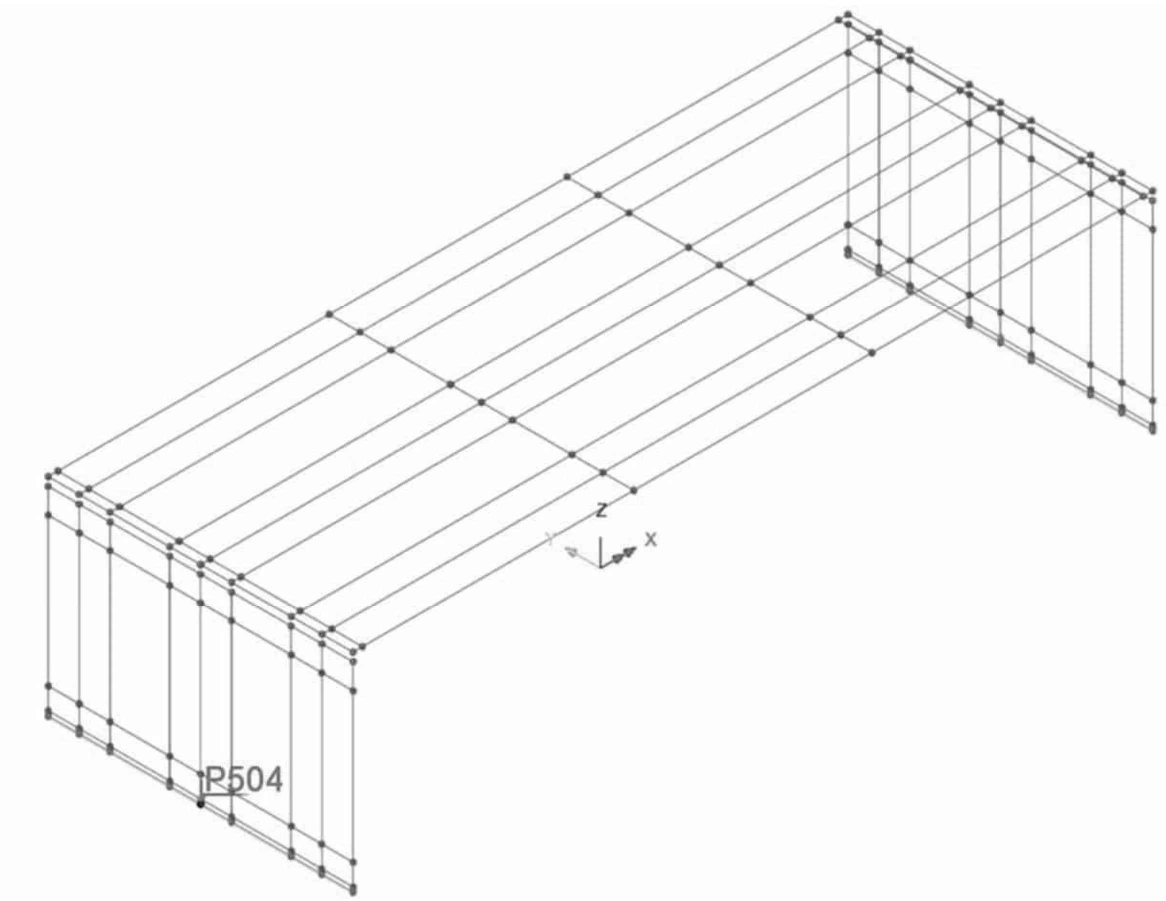
Load: STOD_1Z

Structural loading : Perscribed Displacement

Translation at point in Z direction : -0.010 m

Loadcase : STOD_1Z

Point : P504



Overview 3D

3.4.1.2 Support 2

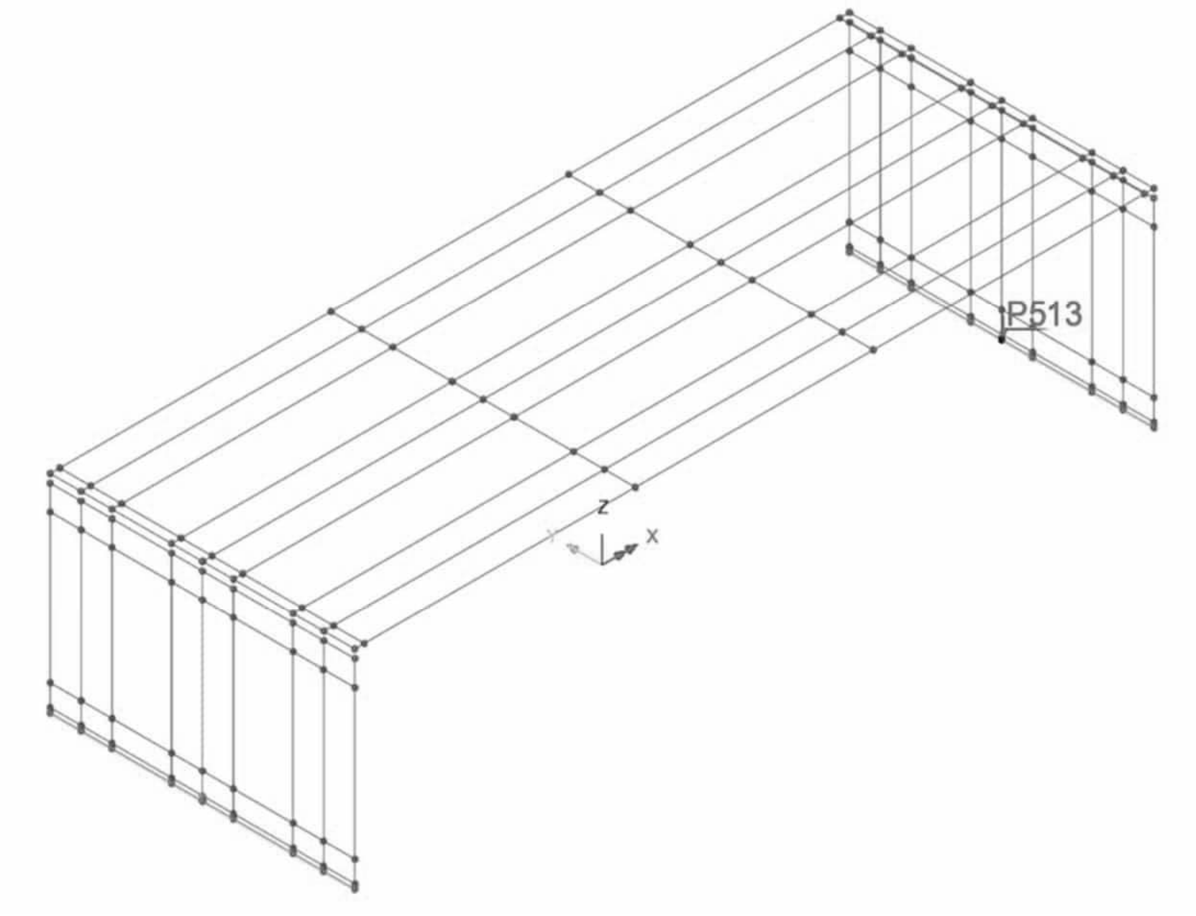
Load: STOD_2Z

Structural loading : Perscribed Displacement

Translation at point in Z direction : -0.010 m

Loadcase : STOD_2Z

Point : P513



Overview 3D

3.4.2 Horizontal settlement

3.4.2.1 Support 1

Load : STOD_1X

Structural loading : Prescribed Displacement

Translation at point in X direction : 0.010 m

Loadcase : STOD_1X+

Point : P504

3.4.2.2 Support 2

Load : STOD_2X

Structural loading : Prescribed Displacement

Translation at point in X direction : 0.010 m

Loadcase : STOD_2X+

Point : P513

3.4.3 Load combination settlement: STOD

Basic load cases :

Load case	Load	Factor
STOD_1X-	STOD_1X+	-1
STOD_2X-	STOD_2X+	-1

Envelope STOD-X :

Load case
STOD_1X+
STOD_2X+
STOD_1X-
STOD_2X-

Envelope STOD-Z :

Load case
STOD_1Z
STOD_2Z

Envelope STOD :

Load case
STOD-X
STOD-Z

3.5 CREEP

Total creep is determined according to SS-EN 1992-1-1 §3.1.4 and TRVINFRA-00227 section 7.1.6.4 for RH 80% at time t_1.

Time for first loading (= time when formwork was removed) is termed t_0.

$t_0 = 5\ days$

$t_1 = 120\ years$

Bridge consists of parts with different thicknesses as seen below.

Creep is determine using Mathcad program PROG A001.

Substructure (b = 12.8 m; C35/45):
For t = 0.80 m → $\phi(t_1, t_0) = 1.95$: see page A3:49
For t = 1.20 m → $\phi(t_1, t_0) = 1.93$: see page A3:49

Superstructure (b = 2.60 m; C40/50):
For t = 1.20 m → $\phi(t_1, t_0) = 1.76$: see page A3:49
For t = 1.70 m → $\phi(t_1, t_0) = 1.73$: see page A3:49

Creep $\phi(t_1, t_0) = 1.70$ is used for the entire bridge on safe side since reduces stiffness and associated constraint forces (∴ support settlement, shrinkage and temperature).

$$\varepsilon_{cc}(t_1, t_0) = \varphi(t_1, t_0) \cdot \frac{\sigma_c}{E_c}$$

To study the effect concrete stiffness according to SS-EN 1992-1-1 5.8.7 creep values seen below are used.

Load cases	φ
Permanent	1.9
Variable excluding temperature	0
Temperature	0.3*

* = According to Swedish work practice

$$E^{system} = \frac{E_{cm}}{1+\varphi}$$

Instead of adjusting E-modulus the load coefficients are adjusted.

$$f_{KRYMP} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+1.70} = 0.37$$

$$f_{STÖD} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+1.70} = 0.37$$

$$f_{JTEMP} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+0.3} = 0.77$$

Note:
According to TRVINFRA-00227 section 7.2.1.1.2.4, no reduction is permitted for uneven temperature across the cross-section. This is because this temperature variation is considered to have a very short duration (only over the day).

Object: Abutment & superstructure

INPUT

Number of sections

$N := 4\ pcs$

Geometry & concrete (C30/37, C35/45, C40/50 & C45/55)

Section	B	H	Concrete
1	12,8	0,8	C30/37
2	12,8	1,2	C30/37
3	2,6	1,2	C40/50
4	2,6	1,7	C40/50
-	m	m	-

Relative humidity

$RH := 80\%$

Time of loading (i.e. removal formwork)

$t_0 := 5\ days$

Studied time for determination of creep

$t_2 := 120\ year$ $t_2 = 43800\ days$

Input receipt

$f_{cm} = [38\ \ 38\ \ 48\ \ 48]\ MPa$

CALCULATION

Area

$$A_c := B \cdot H$$

Perimeter exposed to "air"

$$u := 2 \cdot B$$

Effective thickness structure

$$h_0 := \frac{2 \cdot A_c}{u}$$

Creep coefficients

The expressions for determining the creep coefficients are taken from SS-EN 1992-1-1 Annex B.1.

$$\alpha_1 := \left(\frac{35 \cdot MPa}{f_{cm}}\right)^{0.7} = \begin{bmatrix} 0.94 \\ 0.94 \\ 0.8 \\ 0.8 \end{bmatrix}$$

$$\alpha_2 := \left(\frac{35 \cdot MPa}{f_{cm}}\right)^{0.2} = \begin{bmatrix} 0.984 \\ 0.984 \\ 0.939 \\ 0.939 \end{bmatrix}$$

$$\alpha_3 := \left(\frac{35 \cdot MPa}{f_{cm}}\right)^{0.5} = \begin{bmatrix} 0.96 \\ 0.96 \\ 0.854 \\ 0.854 \end{bmatrix}$$

$$\varphi_{RH} := \begin{cases} \text{if } f_{cm} \le 35\ MPa \\ \quad \varphi_{RH} \leftarrow 38 \cdot MPa \\ \text{else} \\ \quad \varphi_{RH} \leftarrow \left(1 + \dfrac{1 - RH}{0.1 \cdot \sqrt[3]{\dfrac{h_0}{mm}}} \cdot \alpha_1\right) \cdot \alpha_2 \end{cases}$$

$$\varphi_{RH} = \begin{bmatrix} 1.184 \\ 1.158 \\ 1.08 \\ 1.065 \end{bmatrix}$$

$$\beta_0 := \frac{1}{0.1 + {t_0}^{0.20}} = 0.68$$

$$\beta_{fcm} := \frac{16.8}{\sqrt{\dfrac{f_{cm}}{MPa}}} = \begin{bmatrix} 2.73 \\ 2.73 \\ 2.42 \\ 2.42 \end{bmatrix}$$

$$\beta_H := \begin{array}{l} \text{if } f_{cm} \le 35 \cdot MPa \\ \quad \beta_{H.max} \leftarrow 1500 \\ \quad \text{if } 1.5 \cdot \left(1 + (0.012 \cdot 100 \cdot RH)^{18}\right) \cdot \dfrac{h_0}{mm} + 250 > \beta_{H.max} \\ \quad\quad \beta_H \leftarrow \beta_{H.max} \\ \quad \text{else} \\ \quad\quad \beta_H \leftarrow 1.5 \cdot \left(1 + (0.012 \cdot 100 \cdot RH)^{18}\right) \cdot \dfrac{h_0}{mm} + 250 \\ \text{if } f_{cm} > 35 \cdot MPa \\ \quad \beta_{H.max} \leftarrow 1500 \cdot \alpha_3 \\ \quad \text{if } 1.5 \cdot \left(1 + (0.012 \cdot 100 \cdot RH)^{18}\right) \cdot \dfrac{h_0}{mm} + 250 > \beta_{H.max} \\ \quad\quad \beta_H \leftarrow \beta_{H.max} \\ \quad \text{else} \\ \quad\quad \beta_H \leftarrow 1.5 \cdot \left(1 + (0.012 \cdot 100 \cdot RH)^{18}\right) \cdot \dfrac{h_0}{mm} + 250 \cdot \alpha_3 \end{array}$$

$$\beta_H = \begin{bmatrix} 1440 \\ 1440 \\ 1281 \\ 1281 \end{bmatrix}$$

$$\beta_c := \left(\frac{t_2 - t_0}{\beta_H + t_2 - t_0}\right)^{0.3} = \begin{bmatrix} 0.99 \\ 0.99 \\ 0.99 \\ 0.99 \end{bmatrix}$$

$$\varphi_{t0} := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_0 = \begin{bmatrix} 2.18 \\ 2.13 \\ 1.77 \\ 1.75 \end{bmatrix}$$

RESULTS

$$\varphi_{t2} := \varphi_{t0} \cdot \beta_c = \begin{bmatrix} 2.16 \\ 2.11 \\ 1.76 \\ 1.73 \end{bmatrix}$$

3.6 SHRINKAGE

Load applied to Analysis : *Analysis 2*

Total shrinkage according to SS-EN 1992-1-1 §3.1.4 and TRVINFRA-00227 section 7.1.6.4 for RH 80% at time t_1.

Determination of load effect from shrinkage should consider the reduced concrete stiffness from creep.

$t_s = 0\ days$

$t_1 = 120\ years$

Shrinkage is determined using Mathcad program PROG A002 after time t_1.

Substructure (b = 12.8 m; C35/45):
For t = 0.80 m → $\varepsilon_{cs}(t_1) = 0.024\%$: see page A3:53
For t = 1.20 m → $\varepsilon_{cs}(t_1) = 0.023\%$: see page A3:53

Superstructure (b = 2.60 m; C40/50):
For t = 1.20 m → $\varepsilon_{cs}(t_1) = 0.025\%$: see page A3:53
For t = 1.70 m → $\varepsilon_{cs}(t_1) = 0.024\%$: see page A3:53

Shrinkage $\varepsilon_{cs} = 0.025\%$ is applied to all construction parts for safety. The movement corresponds to that which occurs due to an imaginary temperature load ∴ T = -25°C.

Remark
Shrinkage must be considered for service state (SLS) see SS-EN 1992-1-1 §2.3.2.2(1).

Shrinkage does not have to be used for ultimate state (ULS) see SS-EN 1992-1-1 §2.3.2.2(2).

Object: Abutment & superstructure

Number of sections

$N := 4\ pcs$

Geometry & concrete (C30/37, C35/45, C40/50 & C45/55)

Section	B	H	Concrete
1	12,8	0,8	C30/37
2	12,8	1,2	C30/37
3	2,6	1,2	C40/50
4	2,6	1,7	C40/50
-	m	m	-

Relative humidity

$RH := 80\%$

Time of load (i.e. removal formwork)

$t_0 := 5 \cdot days$

Studied time for determination of shrinkage

$t_2 := 120\ year$ $\qquad$ $t_2 = 43800\ days$

Cement class (S, N, R)

$Klass := \text{"N"}$

Concrete age when drying starts

$t_s := 0 \cdot days$

Input receipt

$f_{cm} = [38\ \ 38\ \ 48\ \ 48]\ MPa$ $\qquad$ $f_{ck} = [30\ \ 30\ \ 40\ \ 40]\ MPa$ $\qquad$ $f_{cmo} = 10\ MPa$

CALCULATION

Area

$A_c := B \cdot H$

Perimeter exposed to "air"

$u := 2 \cdot B$

Effective thickness structure

$$h_0 := \frac{2 \cdot A_c}{u} = \begin{bmatrix} 0.8 \\ 1.2 \\ 1.2 \\ 1.7 \end{bmatrix} m$$

Basic value of drying shrinkage see SS-EN 1992-1-1, Annex B.2

$$\alpha_{ds1} := \begin{array}{l} \text{if } Klass = \text{"S"} \\ \quad 3.0 \\ \text{if } Klass = \text{"N"} \\ \quad 4.0 \\ \text{if } Klass = \text{"R"} \\ \quad 6.0 \end{array} = 4.00$$

$$\alpha_{ds2} := \begin{array}{l} \text{if } Klass = \text{"S"} \\ \quad 0.13 \\ \text{if } Klass = \text{"N"} \\ \quad 0.12 \\ \text{if } Klass = \text{"R"} \\ \quad 0.11 \end{array} = 0.12$$

$RH_o := 100\%$

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_o}\right)^3\right) = 0.76$$

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}}\right) \cdot 10^{-6} \cdot \beta_{RH} = \begin{bmatrix} 2.69 \cdot 10^{-4} \\ 2.69 \cdot 10^{-4} \\ 2.385 \cdot 10^{-4} \\ 2.385 \cdot 10^{-4} \end{bmatrix}$$

Basic drying shrinkage (SS-EN 1992-1-1, section 3.1.4, see equations 3.9 and 3.1

$$k_h := \text{linterp}\left(\left(\begin{bmatrix} 0 & 100 & 200 & 300 & 500 & 10^4 \end{bmatrix} \cdot mm\right), \begin{bmatrix} 1.00 & 1.00 & 0.85 & 0.75 & 0.70 & 0.70 \end{bmatrix}, h_0\right) = \begin{bmatrix} 0.70 \\ 0.70 \\ 0.70 \\ 0.70 \end{bmatrix}$$

$$\beta_{ds} := \frac{t_2 - t_s}{t_2 - t_s + 0.04 \cdot \sqrt{\left(\frac{h_0}{mm}\right)^3}} = \begin{bmatrix} 0.98 \\ 0.96 \\ 0.96 \\ 0.94 \end{bmatrix}$$

$$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = \begin{bmatrix} 1.845 \cdot 10^{-4} \\ 1.814 \cdot 10^{-4} \\ 1.609 \cdot 10^{-4} \\ 1.569 \cdot 10^{-4} \end{bmatrix}$$

Autogenous-shrinkage, see EN 1992-1-1 §3.1.4, eqns. 3.11–3.13

$$\beta_{as} := 1 - e^{-0.2 \cdot \sqrt{t_2}} = 1.00$$

$$\varepsilon_{ca.\infty} := 2.5 \cdot \left(\frac{f_{ck}}{MPa} - 10\right) \cdot 10^{-6} = \begin{bmatrix} 5 \cdot 10^{-5} \\ 5 \cdot 10^{-5} \\ 7.5 \cdot 10^{-5} \\ 7.5 \cdot 10^{-5} \end{bmatrix}$$

$$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca.\infty} = \begin{bmatrix} 5 \cdot 10^{-5} \\ 5 \cdot 10^{-5} \\ 7.5 \cdot 10^{-5} \\ 7.5 \cdot 10^{-5} \end{bmatrix}$$

RESULTS

Total shrinkage, see SS-EN 1992-1-1 §3.1.4, eqn. 3.8

$$\varepsilon_{cs} := \varepsilon_{cd} + \varepsilon_{ca} = \begin{bmatrix} 2.345 \cdot 10^{-4} \\ 2.314 \cdot 10^{-4} \\ 2.359 \cdot 10^{-4} \\ 2.319 \cdot 10^{-4} \end{bmatrix}$$

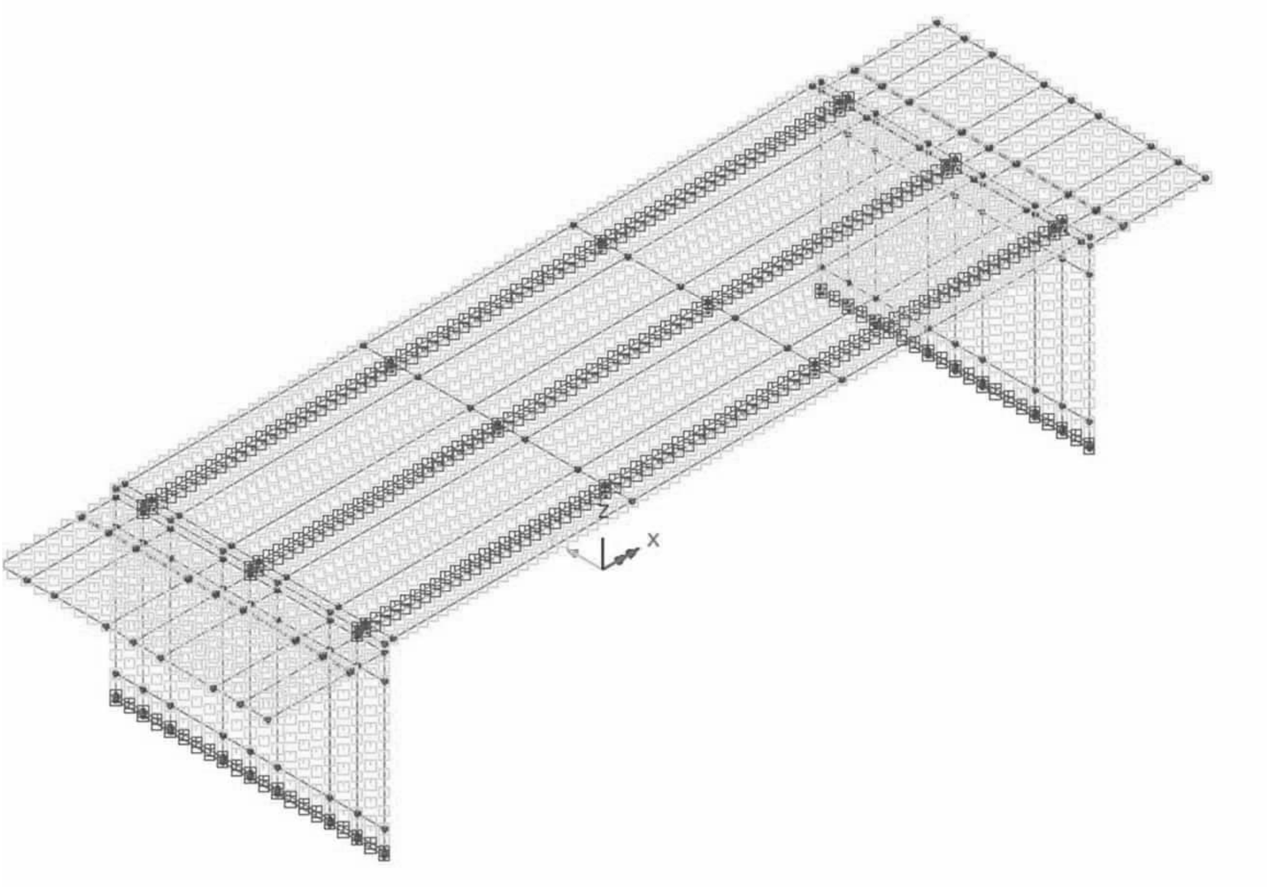
Loadcase : KRYMP

Structural loading : Temperature

Definition : Nodal

Initial temperature : 0 °C

Final temperature : -25 °C



Overview 3D

Remark
Load is applied to entire structure.

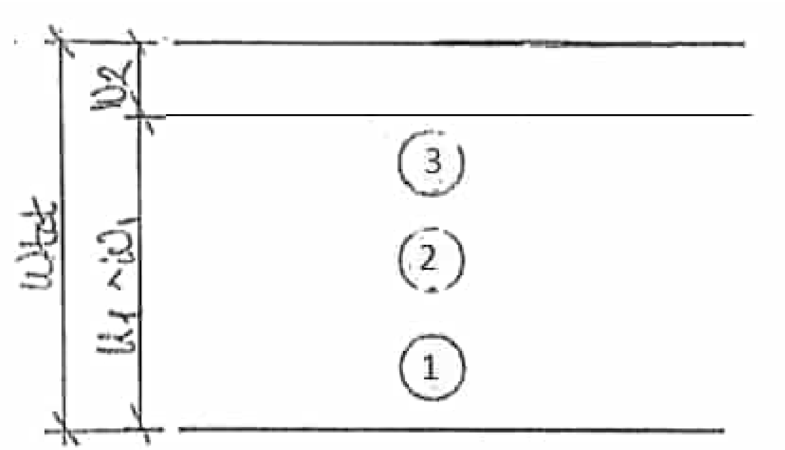
3.7 TRAFFIC

Load applied to Analysis : *Analysis 2*

Evaluation of vertical traffic is performed for LM 1 and LM 2 according to SS-EN 1991-2 section 4.3.

Evaluation will also be performed EG A/B = 180kN/300 kN according to TRVFS 2011:12 chapter 6 point 3§.

3.7.1 Traffic lane division



Total traffic width : $w_{tot} = 12.8\,m$

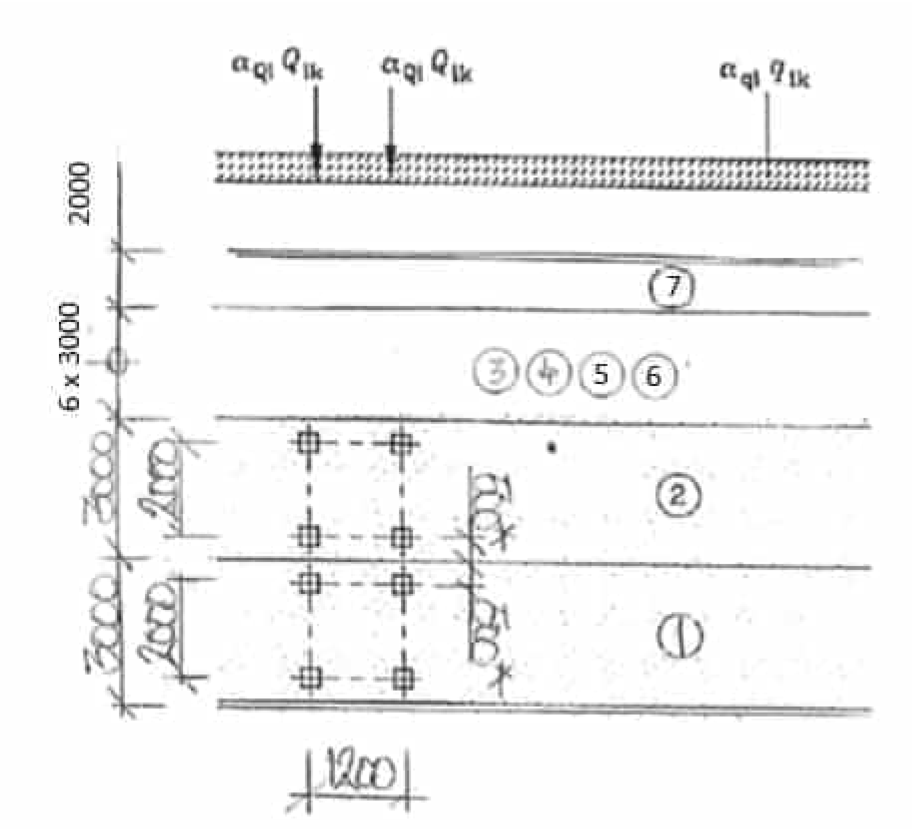
Number of traffic lanes : $n_1 = Integer\left\lfloor\frac{w_{tot}}{3.0m}\right\rfloor = 4\ lanes$

Full traffic width : $w_1 = 3.0m$

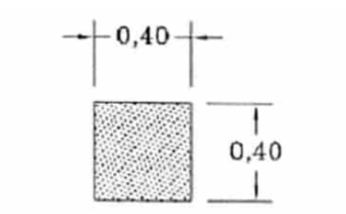
Remaining width : $w_2 = 0.8m$

3.7.2 Load model 1 (LM 1)

Characteristic values according to SS-EN 1991-2 §4.3.2.



* = When studying local effects 250 mm is to be assumed.



Axellaster :

α_Q : national adaptation factor according to TRVFS 2011:12 table 7.1

$Q'_k = \alpha_Q \cdot Q_k$: characteristic value including national adaptation factor

Traffic lane	Q_k	α_Q	Q'_k	Remark
1	300	0,9	270	LM1- 2 x 270 kN
2	200	0,9	180	LM1- 2 x 180 kN
3-6	100	0	0	No load
-	kN	-	kN	-

Utbredda laster :

α_q : national adaptation factor according to TRVFS 2011:12 table 7.1

$q'_k = \alpha_q \cdot q_k$: characteristic value including national adaptation factor

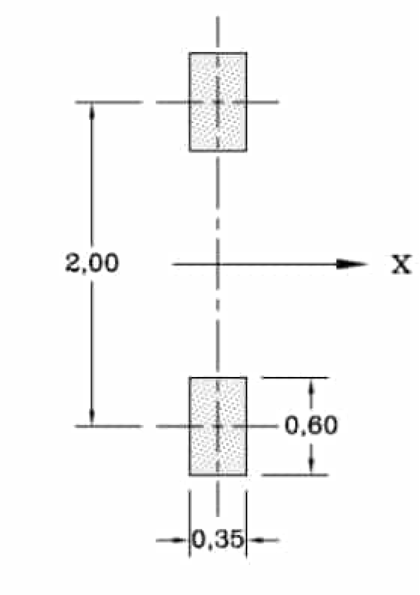
Traffic lande	q_k	α_q	q'_k
1	9.0	0.8	7.2
2-7	2.5	1.0	2.5
-	kPa	-	kPa

Remark

Evaluation is performed using Vehicle Load Optimisation (VLO), see section 3.7.4.

3.7.3 Load model 2 (LM 2)

Characteristic vertical load according to SS-EN 1991-2 §4.3.3.



$\beta_Q = \alpha_Q = 0.90$: national adaptation factor

$Q_k = 400$ kN : characteristic value

$Q'_k = \beta_k \cdot Q_k = 360$ kN : characteristic value including national adaptation factor

Tire pressure

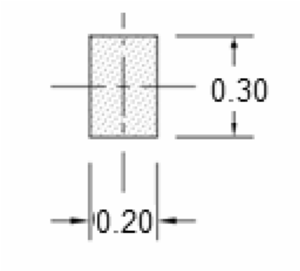
TSFS Chapter 11 Section 4 states that the same contact surface as LM 1 may be used.

3.7.4 Load model EG A/B

Calculation is performed using traffic load EG A/B = 180 kN/300 kN excluding dynamic factor.

Traffic load EG A/B are applied to two traffic lanes. Traffic on first lane is multiplied by 1.00 while second lane is multiplied 0.80.

The center distance between the wheel pressures is 2.0 meters according to TSFS chapter 11 §2.



Wheel pressure

$\varepsilon_{dyn} = 25$ % : dynamic factor [1.)]

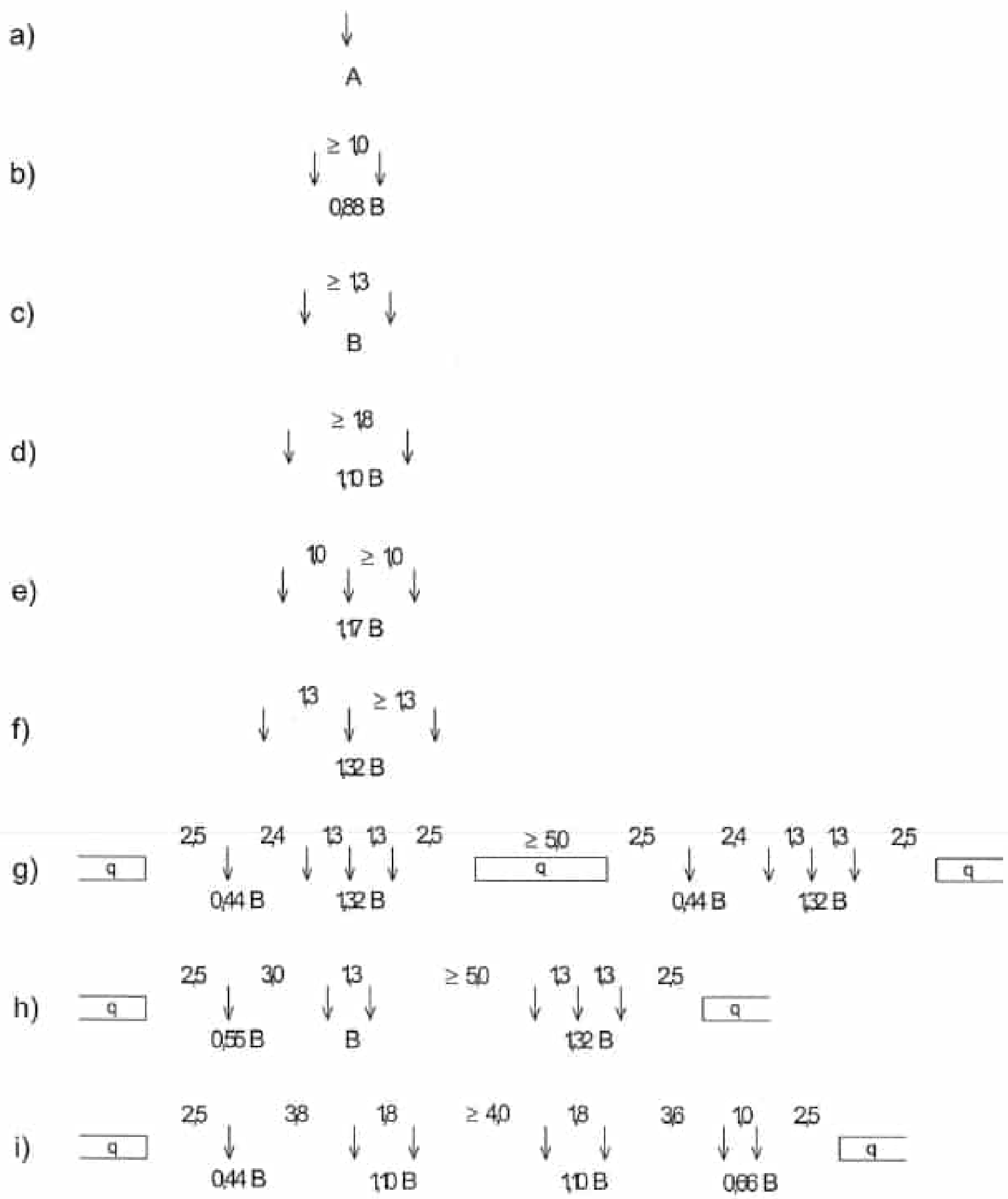
$A' = A \cdot (1 + \varepsilon_{dyn}) = 180$ kN$\cdot(1+0.25) = 225$ kN : single load including dynamic factor

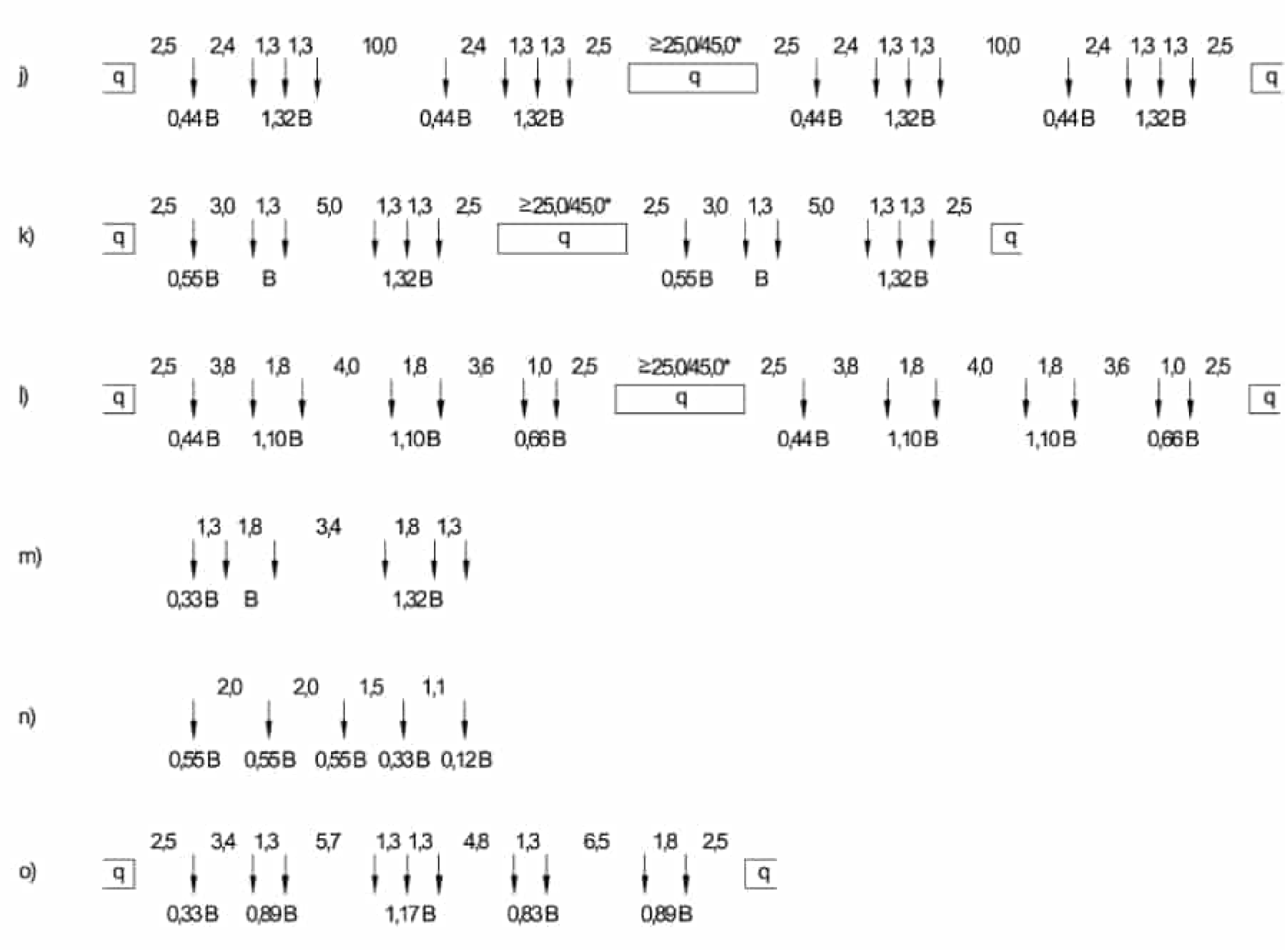
$B' = B \cdot (1 + \varepsilon_{dyn}) = 300$ kN$\cdot(1+0.25) = 375$ kN : tandem load including dynamic factor

$p = 5 \frac{kN}{m}$: surface load

Footnote:
[1.)] TRVINFRA-00227 table 7.1-5 section 4.2.1(1) states apply 25 % ..

Graphic presentation of common vehicle types:
(Vehicle types according to TRVINFRA-00331 Appendix 1)





<u>Note:</u>

Evaluation is carried out with the script Vehicle Load Optimization (VLO), see sections 3.5.3 and 3.5.4.

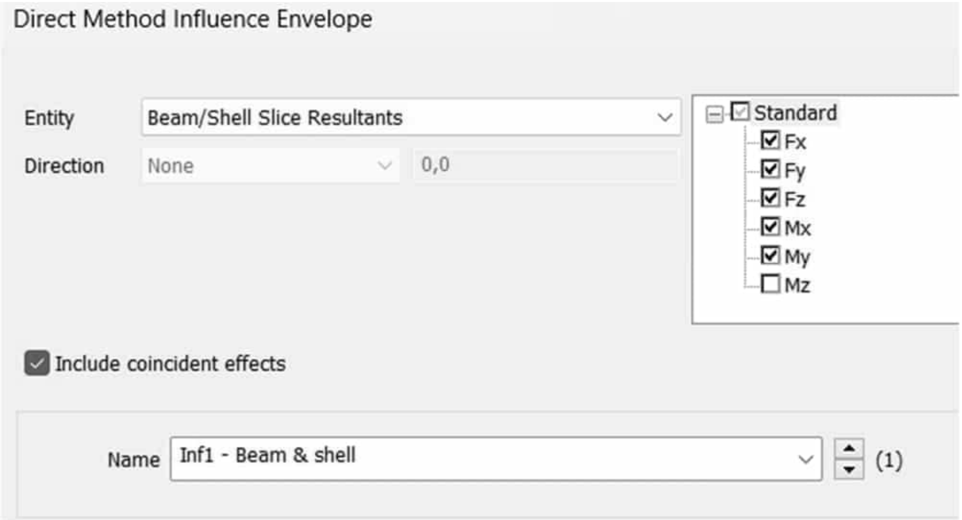
Since there is no motorway, * = 45 m is applied according to TRVINFRA-00331 section 8.3.2.2.1 for vehicle types j, k, and l.

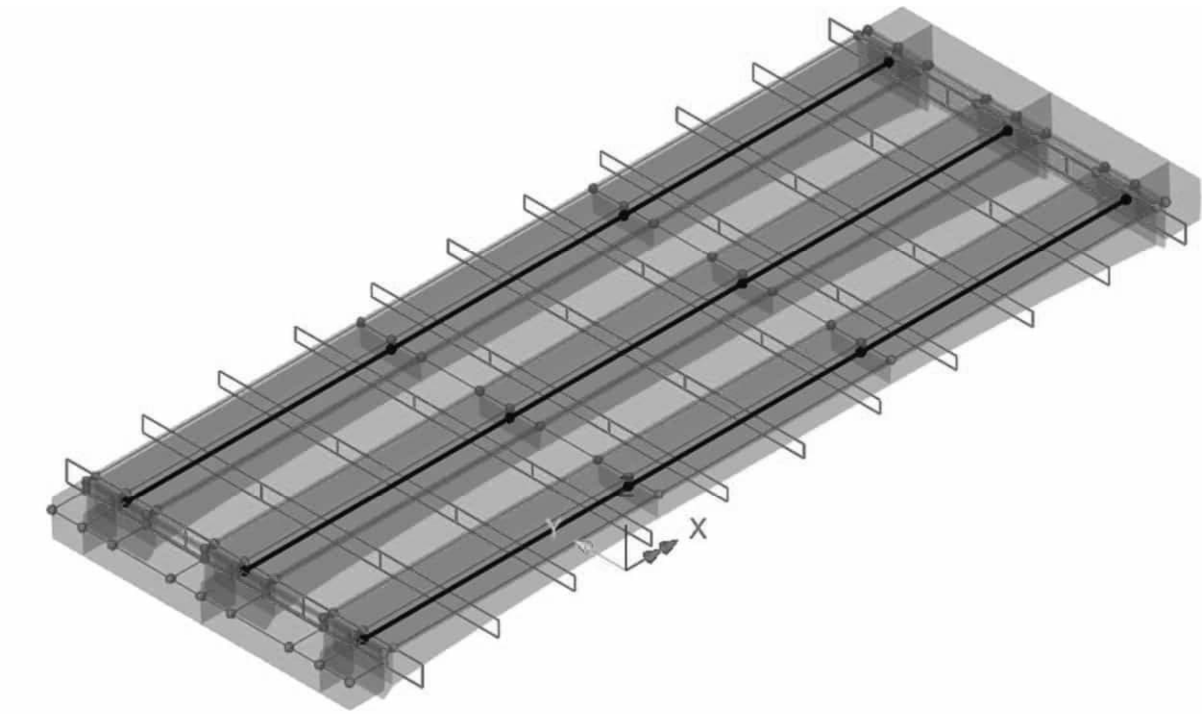
3.7.5 Vehicle Load Optimization (VLO)

3.7.5.1 Influence components

Influence surfaces are created using *Direct Method Influence Envelope.* This is done by applying *Influence components* seen below.

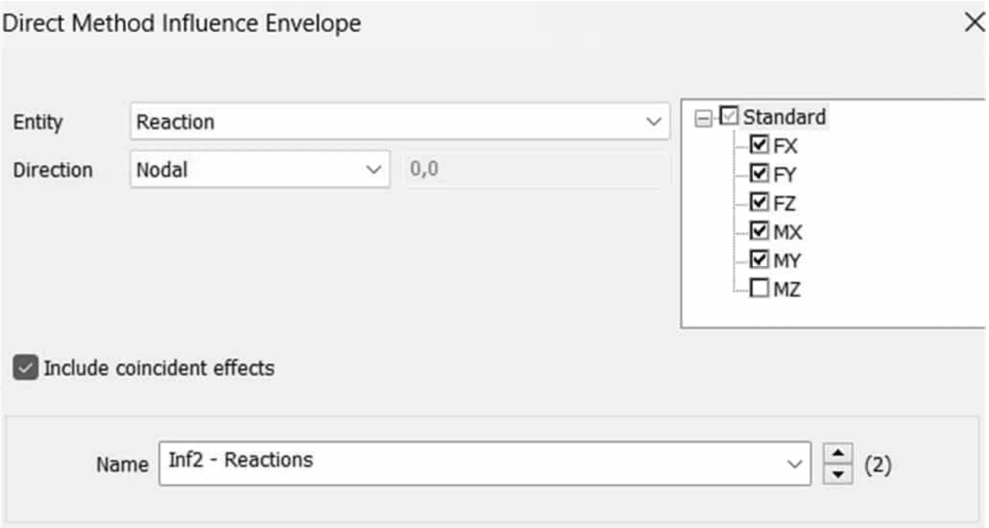
Inf1 – Beam & shells :

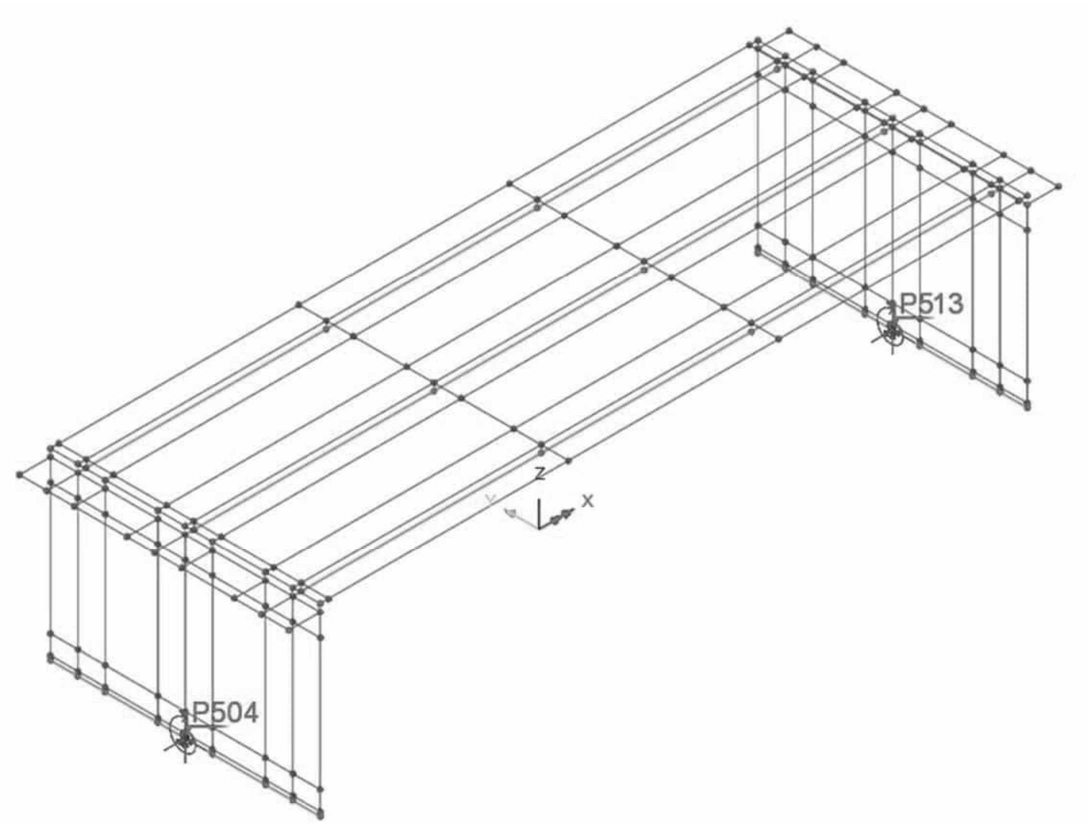




<u>Overview 3D</u>

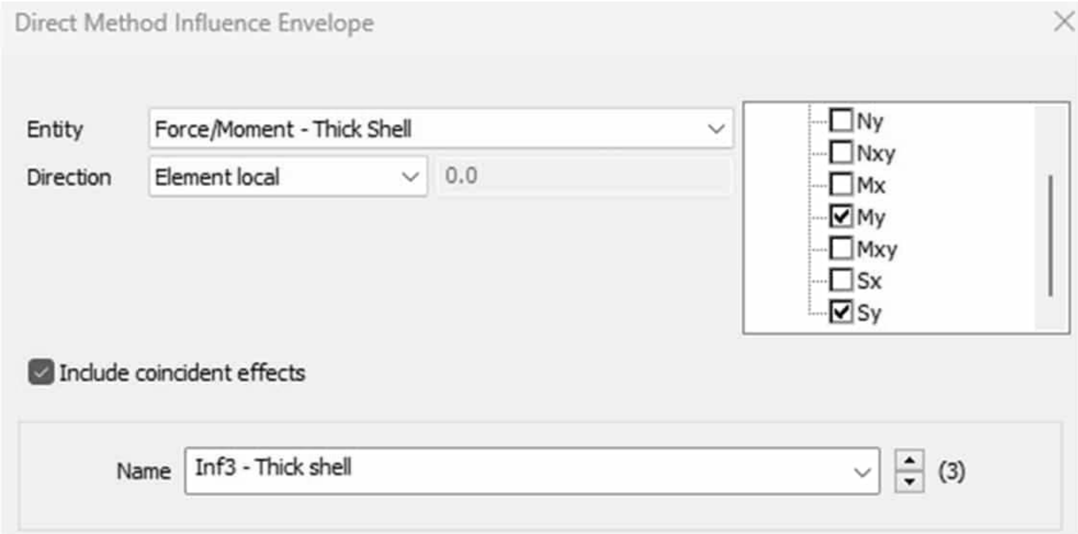
<u>Inf2 – Reactions :</u>

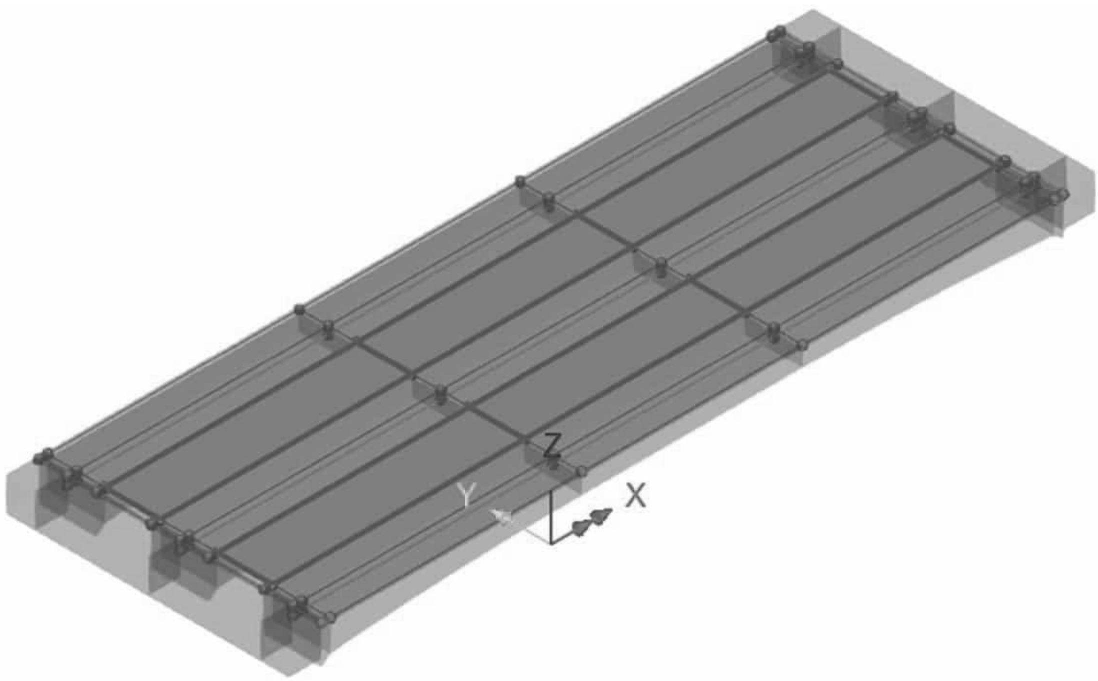




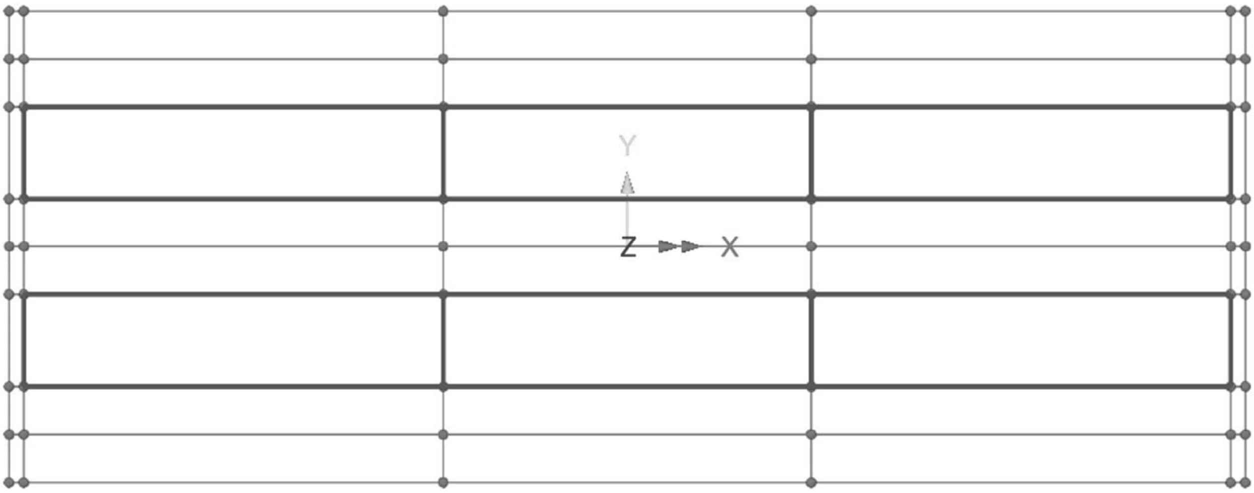
<u>Overview 3D</u>

Inf3 – Thick shells :
(Deck superstructure)



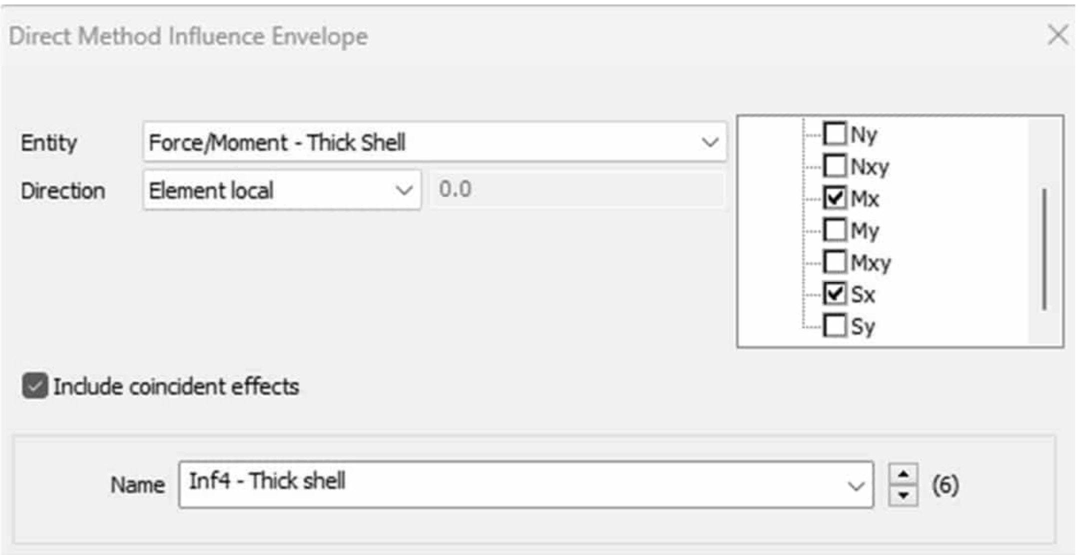


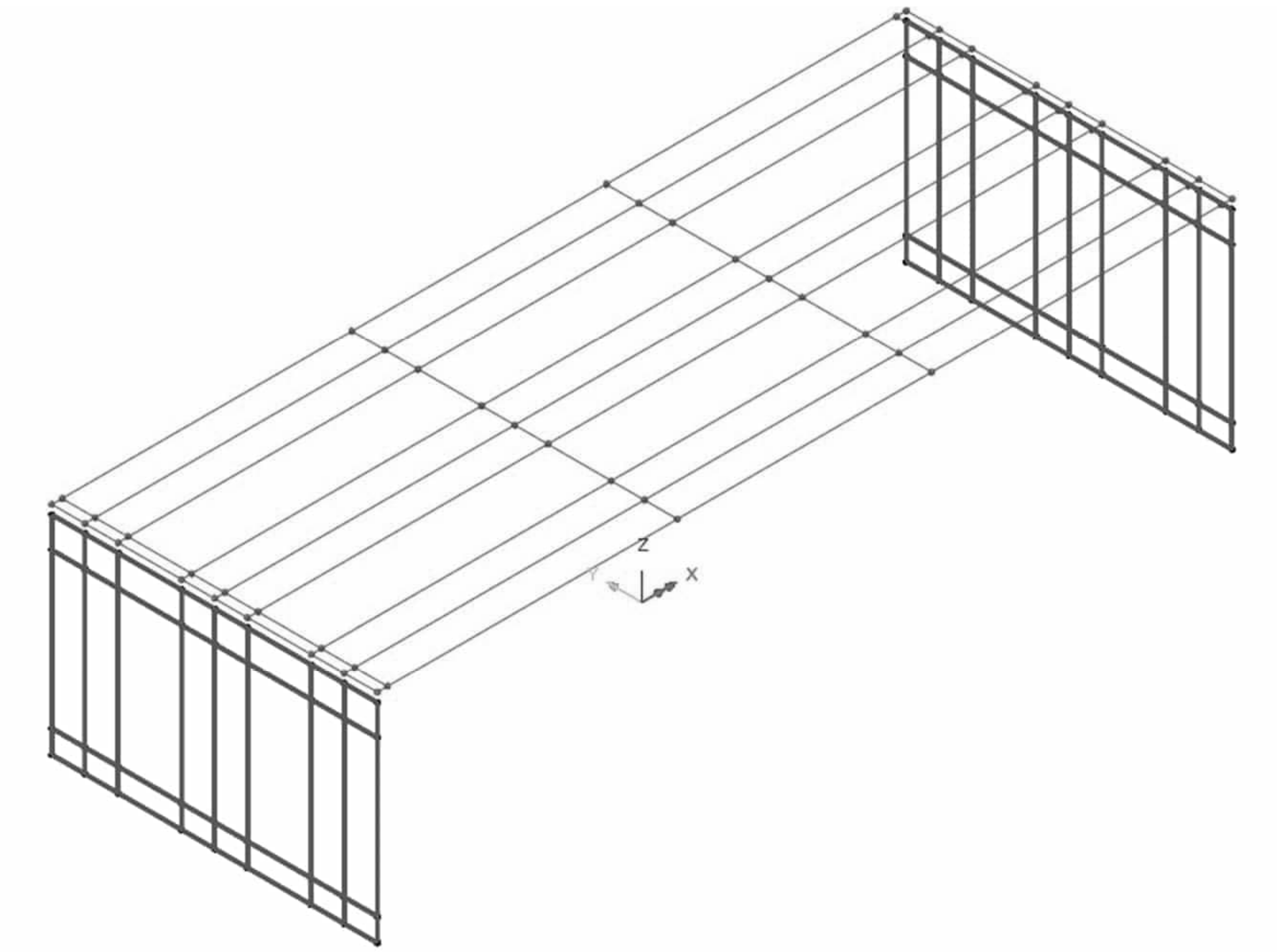
Overview 3D



PLAN

Inf4 – Thick shells :
(Abutments)





Overview 3D

3.7.5.2 Influence surface analysis

Influence surfaces :

Search area: Superstructure

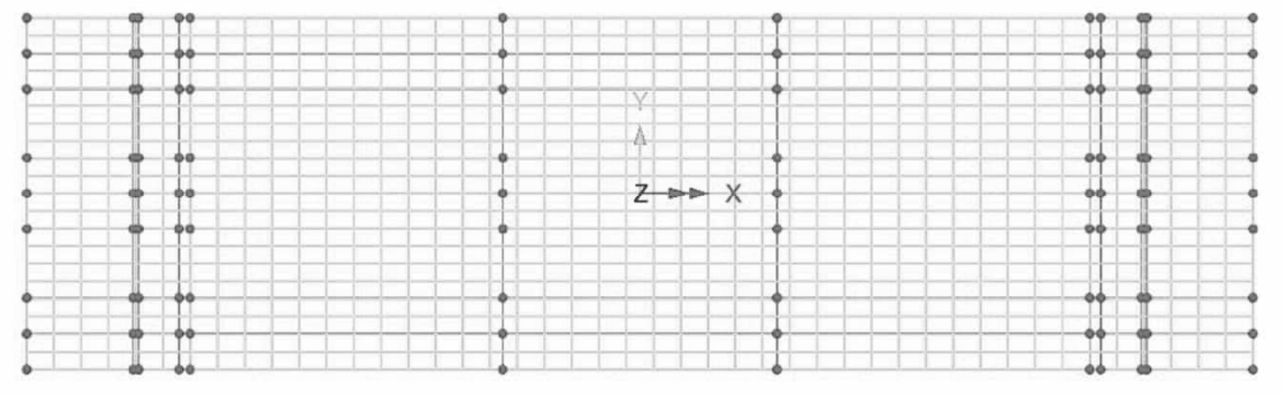
Definition type: Grid

Path: Centerline X

Transverse width: 12.8 m

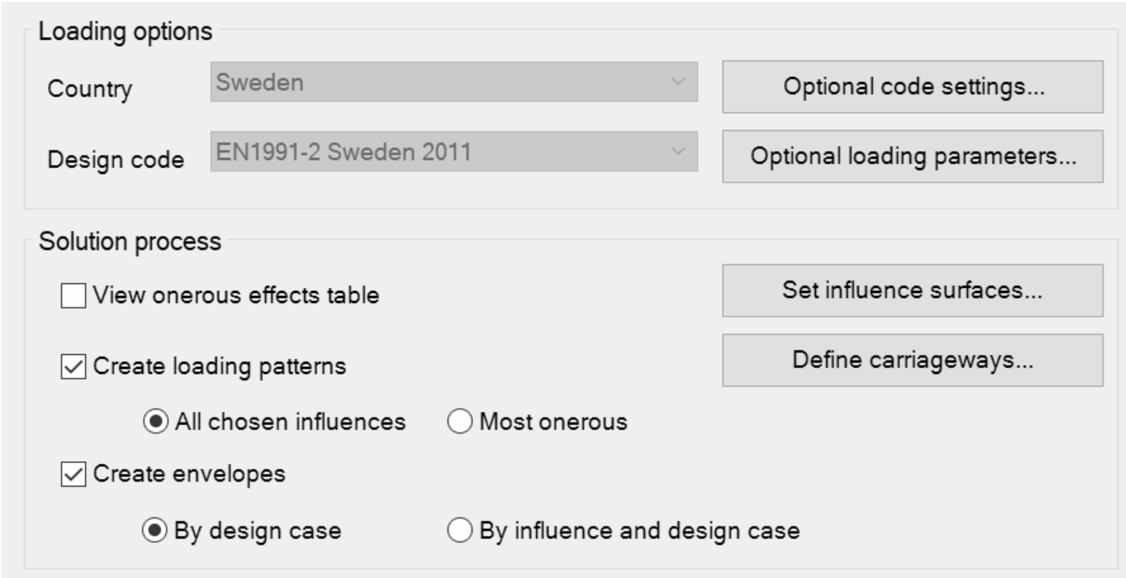
Longitudinal spacing: 1.0 m

Transversal spacing: 0.64 m



PLAN

3.7.5.3 Traffic load analysis (VLO)



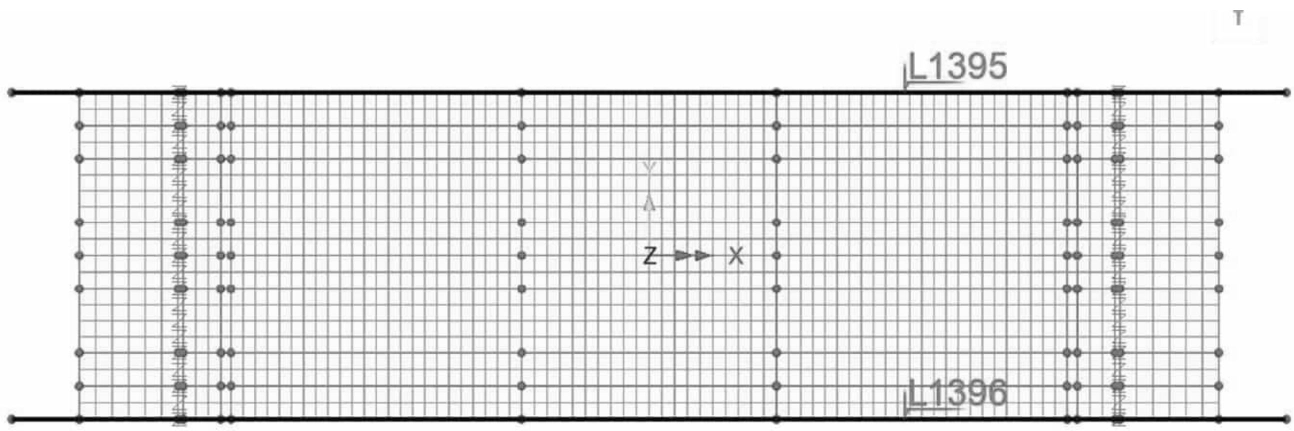
Vehicle longitudinal incremental movement: 0.25 m

Vehicle transverse incremental movement: 0.50 m

Vehicle direction: both

Definition of carriageway (kerbs): L1395 & L1396

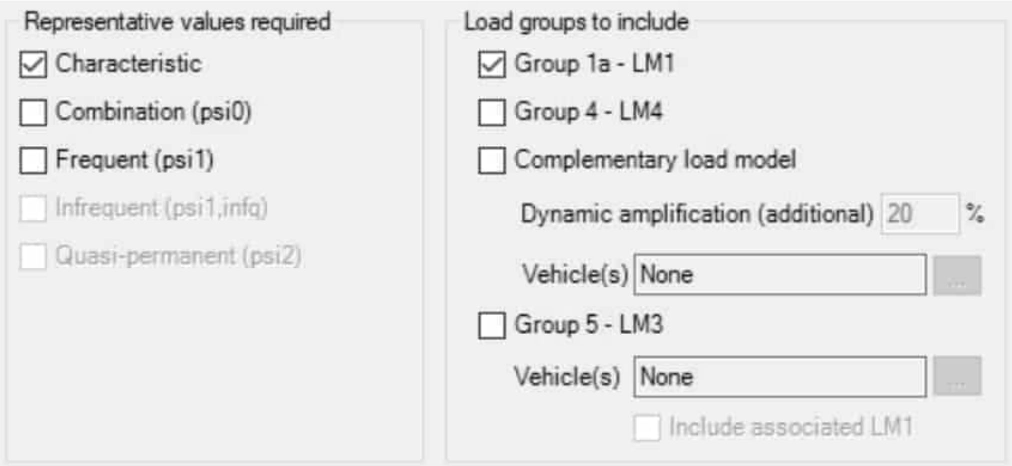
Influence surfaces: Include all (positive & negative)



PLAN

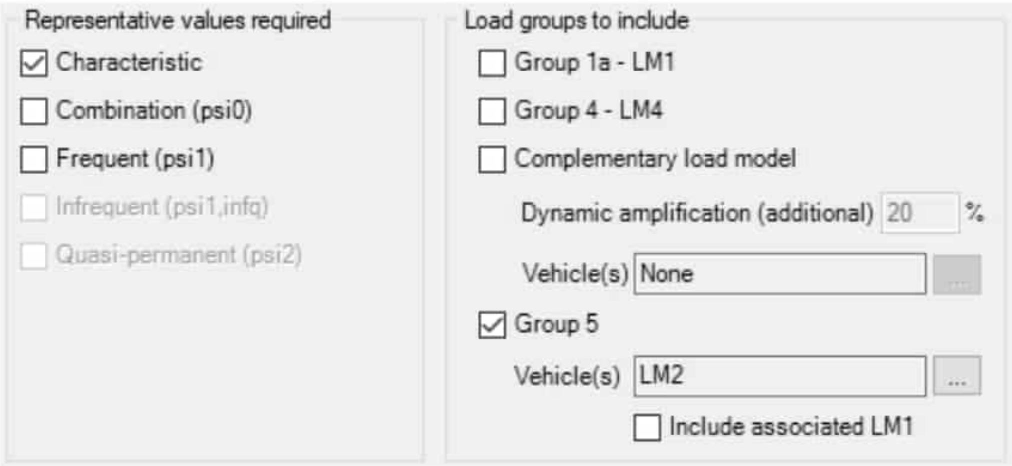
3.7.5.4 Envelope : LM 1

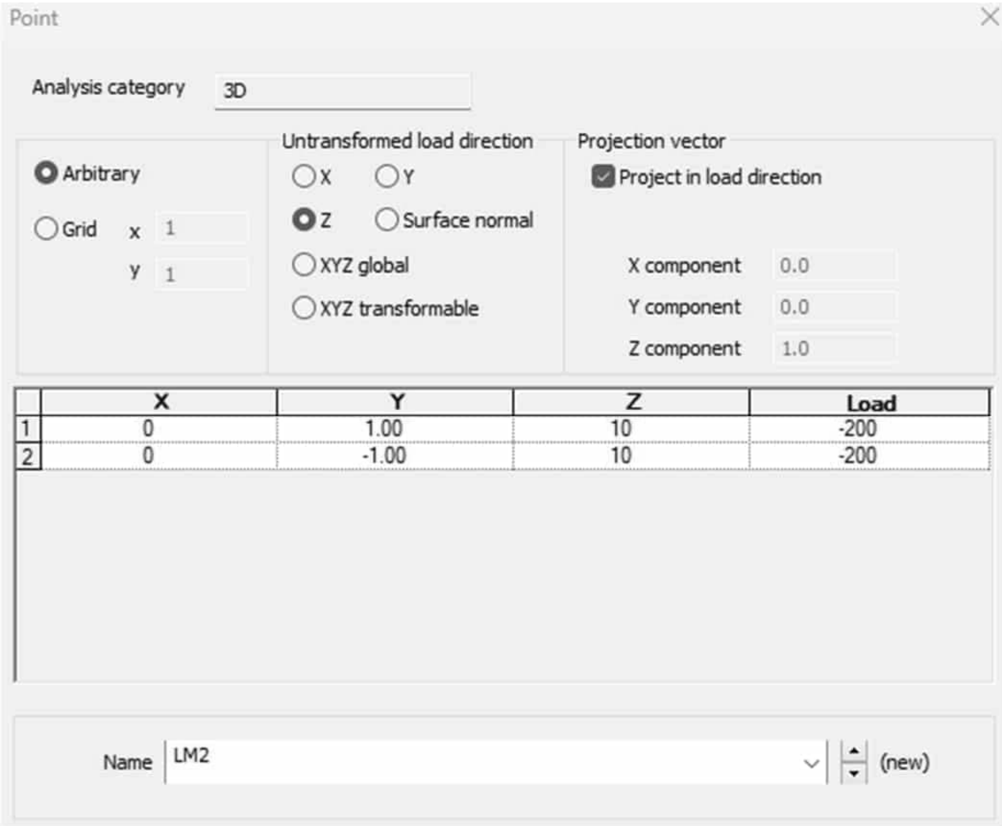
Load model 1 (LM1) defined in SS-EN 1991-2 section 4.3.2.



3.7.5.5 Envelope : LM 2

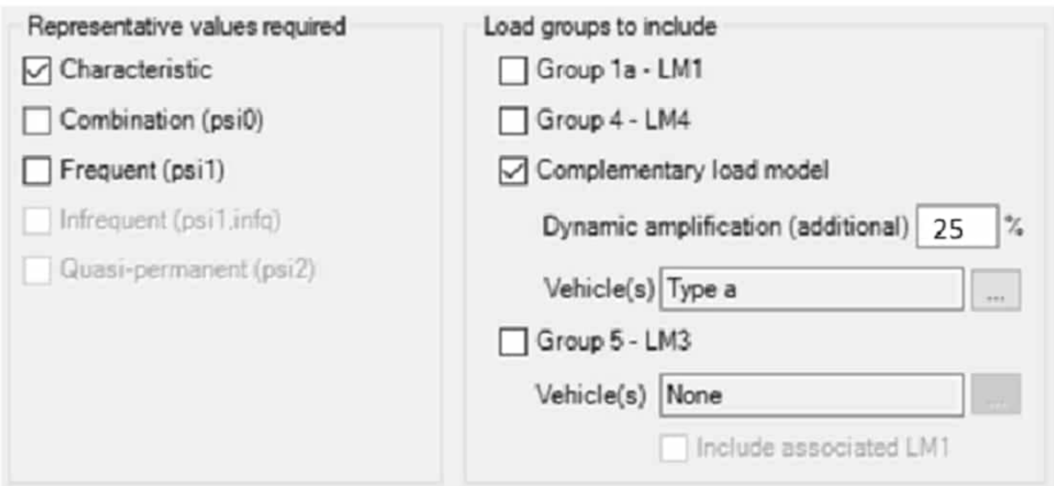
Load model 2 (LM2) defined in SS-EN 1991-2 section 4.3.3. The load is defined in Group 5 vehicle) since Group 1b is not defined in present version of software.





3.7.5.6 Envelope : EG A

EG A is defined as complementary load model with options seen below.

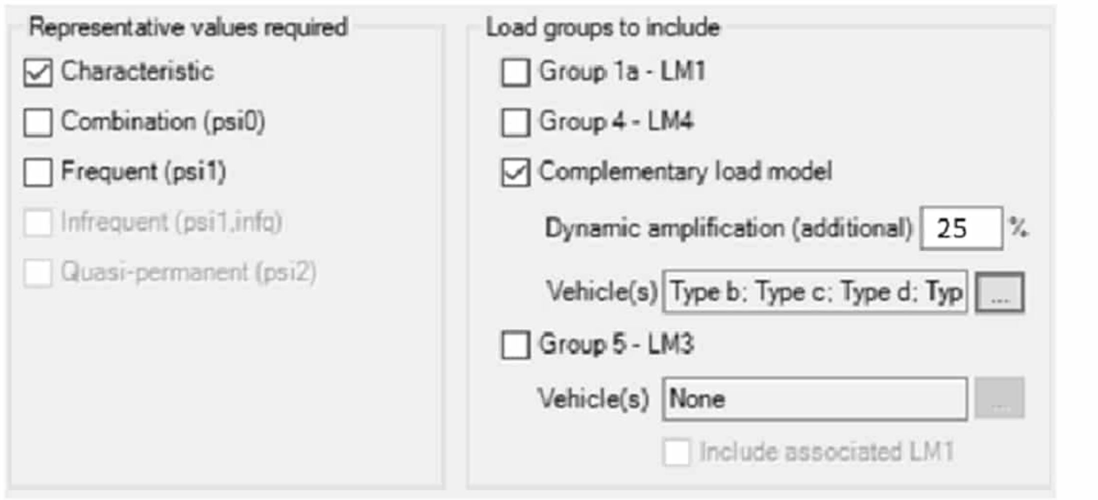


Dynamic amplification (additional): 25 %

Vehicle selection: Type a

3.7.5.7 Envelope : EG B

EG B is defined as complementary load model with options seen below.



Dynamic amplification (additional): 25 %

Vehicle selection: Type b → o

3.7.5.8 Combined traffic load (TRAFIK)

There are a total 4 different traffic loads termed LM 1, LM2, EG A and EG B.

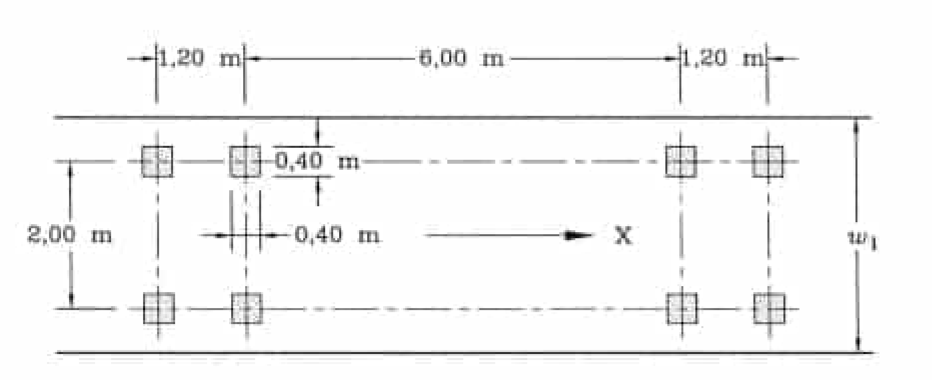
The envelope is used to identify the most onerous load effect.

Envelope TRAFIK :

Envelope
LM 1
LM 2
EG A
EG B

3.7.5.9 Fatigue model

Fatigue model 3 (UTM3) defined in SS-EN 1991-2 section 4.6.4. The load is defined in Group 5 (special vehicle) in present version of software.



Q_k = 120 kN : characteristic value including nation adaptation factors.

Traffic data according to document 1C070001 section B1:

· Reference speed: 80 km/h

· Annual Average Daily Traffic (AADT until 2045) amounts to 7000 vehicles/day

· Proportion of heavy annual average daily traffic is 12% of AADT → 840 vehicles/day until 2045.

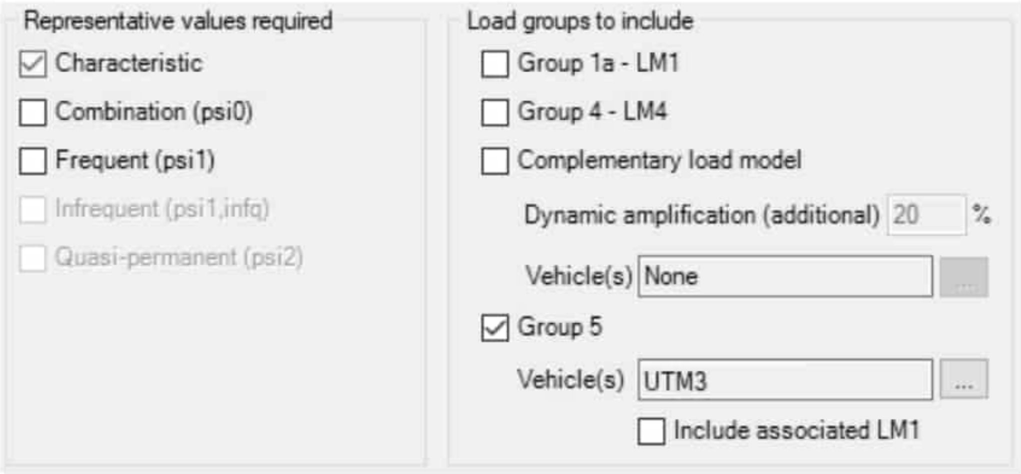
An annual traffic increase of 1% over 100 years results in a total annual AADT of 143,204 vehicles → average AADT = 1,432 vehicles/day.

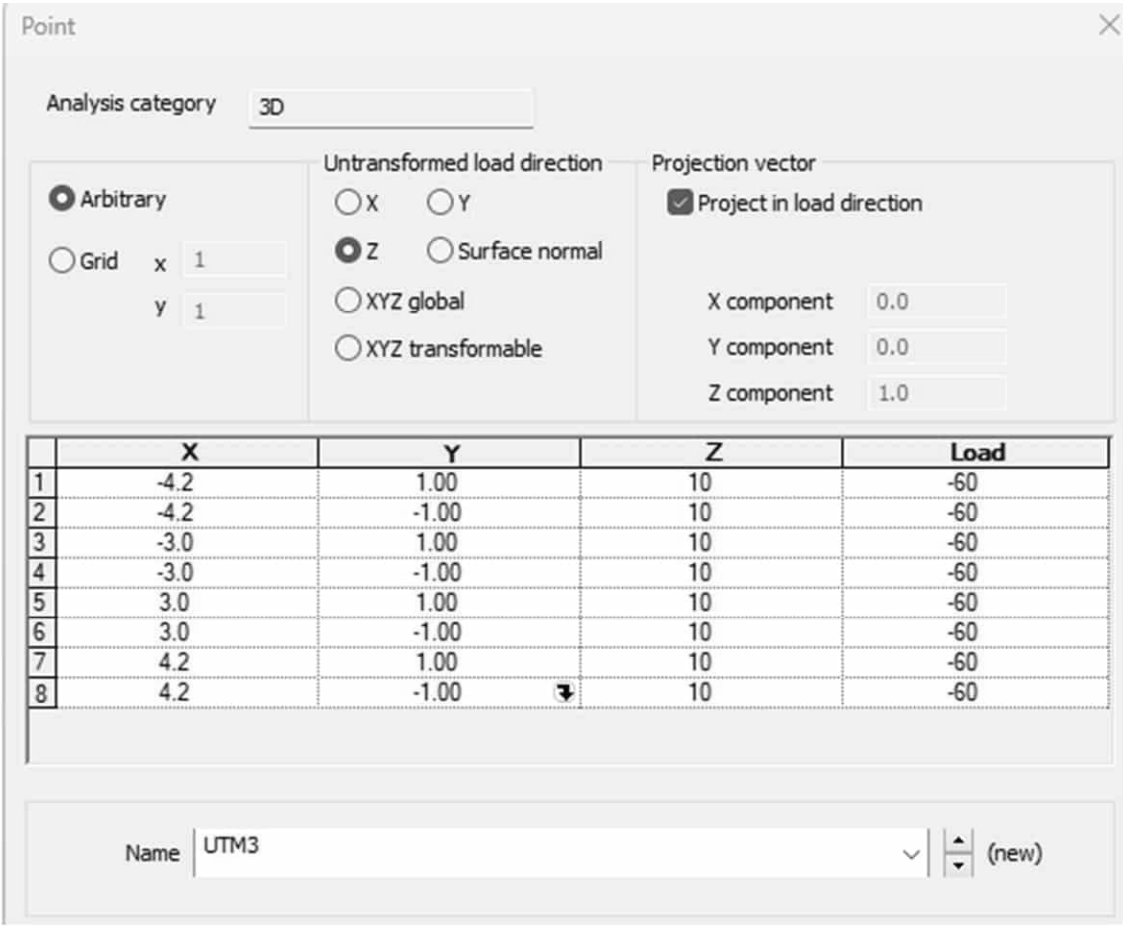
Traffic category:
TRVINFRA-0027 table 7.1-5(h) gives traffic category 3

Reference values for the number of heavy vehicles:
According to SS-EN 1991-2 section 4.6.1 table 4.5(n), Category 3 is obtained

→ N_{obs} = 125,000 vehicles/year

The load definition:
The load UTM3 is defined as a special vehicle in "load group 5."





	X	Y	Z	Load
1	-4.2	1.00	10	-60
2	-4.2	-1.00	10	-60
3	-3.0	1.00	10	-60
4	-3.0	-1.00	10	-60
5	3.0	1.00	10	-60
6	3.0	-1.00	10	-60
7	4.2	1.00	10	-60
8	4.2	-1.00	10	-60

3.8 BRAKING LOAD

Load applied to Analysis : *Analysis 1*

Braking load is defined by SS-EN 1991-2 §4.4.1.

Load acts at level of surfacing.

L = 1.75 m + 32.8 m + 1.75 m = 36.3 m

Load modell LM 1 :

$Q_{lk} = 0.6\alpha_{Q1} \cdot (2Q_{ik}) + 0.1\alpha_{q1} \cdot q_{1k} \cdot w_1 \cdot L$

$180kN \cdot \alpha_{Q1} \leq Q_{lk} \leq 900kN$

$Q_{broms} = 0.6 \cdot (2 \cdot 270kN) + 0.1 \cdot 7.2kPa \cdot 3.0m \cdot 36.3m = 324kN + 78kN = 402kN$

Load modell EG B = 300 kN (see TSFS chapter 11 §2) :

Typ o is dimensioning.

$Q_{lk} = 0.35 \cdot \sum Q_{EG\ B} + 0.1 \cdot p \cdot L_q$

$Q_{lk} \leq 500kN$

$Q_{broms} = 0.35 \cdot (0.89 + 1.17 + 0.83 + 0.89) \cdot B = 0.35 \cdot 3.78 \cdot 300kN = 397kN$

Note:
The braking force associated with LM 1 is applied on the safe side in the system calculation.

The impact of the resisting earth pressure against the frame legs is neglected on the safe side.

3.8.1 Load definition

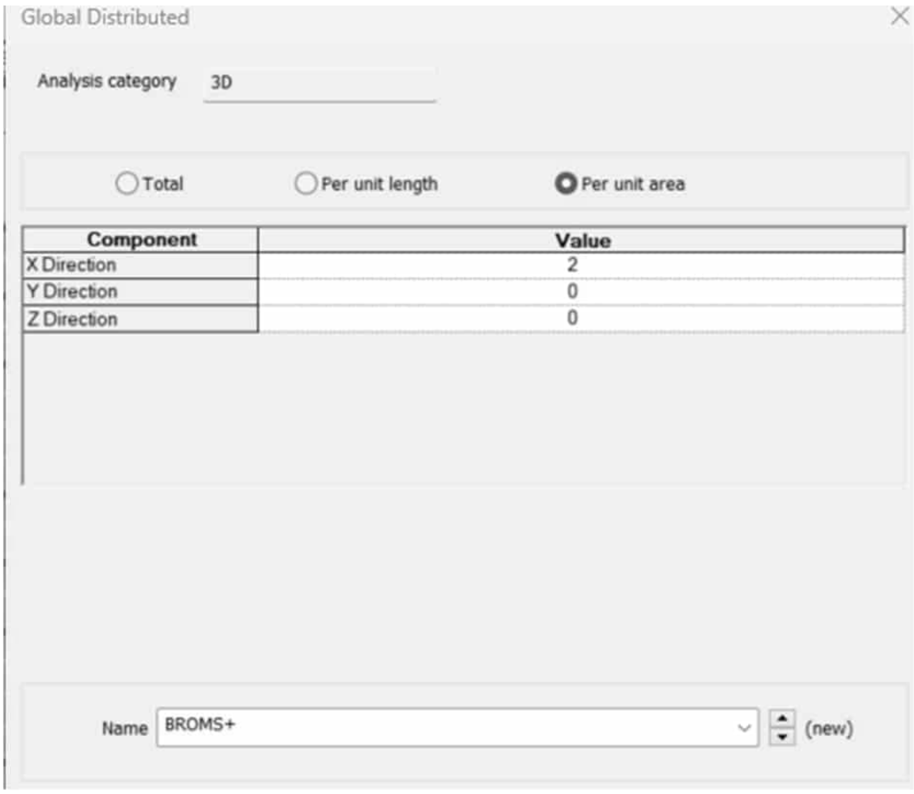
The load is introduced as a surface load in the bridge deck's system line, located 0.25 meters below the pavement level. In the static model, this is disregarded since the effect of load my is small.

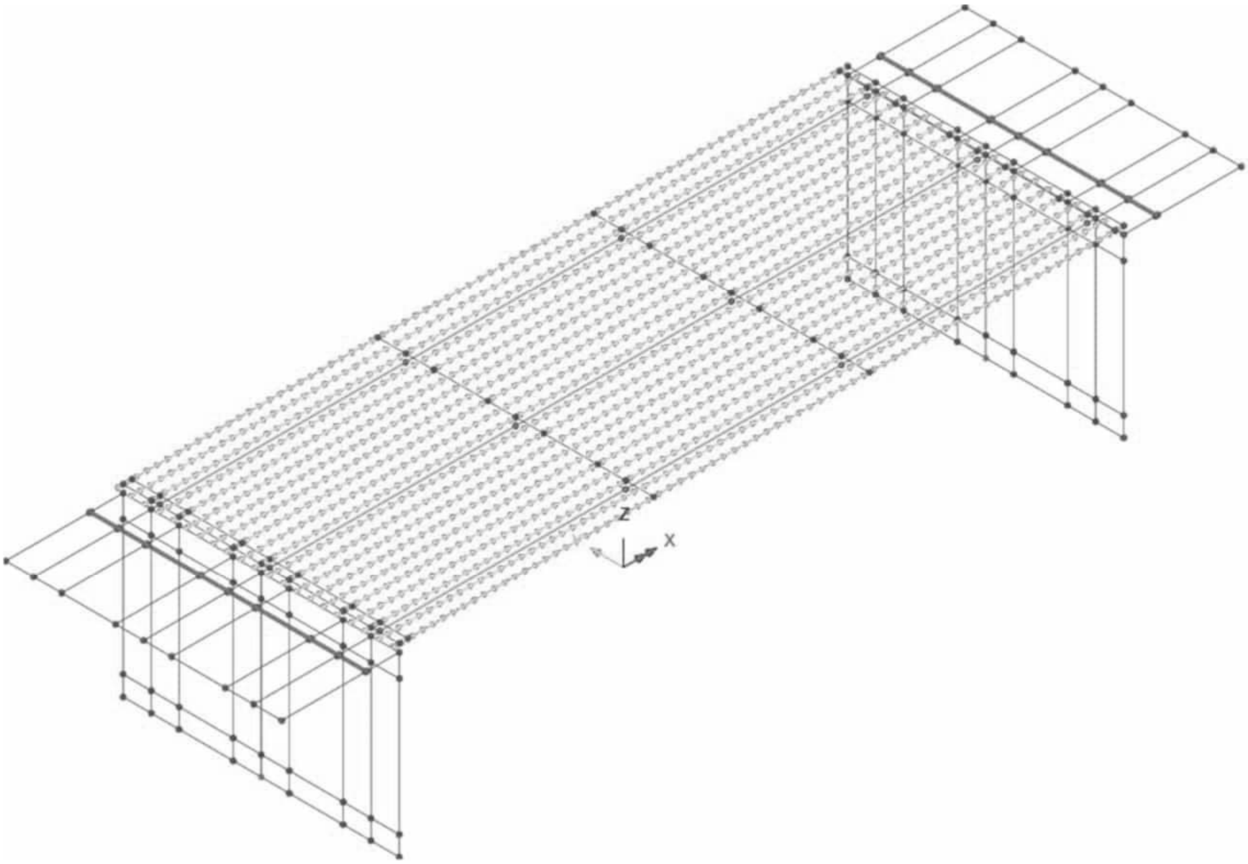
To avoid considering varying load placement laterally, it is assumed that two braking forces occur symmetrically on the bridge deck. This simplification of braking forces is considered safe.
The braking force is considered evenly distributed over the entire bridge deck.

$$q_x = 2 \cdot \frac{Q_{broms}}{w_{tot} \cdot L_{deck}} = 2 \cdot \frac{402kN}{12.8m \cdot 33.6m} = 1.9\frac{kN}{m^2}$$

$$m_y = q_x \cdot (0.15m + t_{bel}) = 1.9\frac{kN}{m^2} \cdot (0.15m + 0.10m) = 0.5\frac{kNm}{m^2}$$

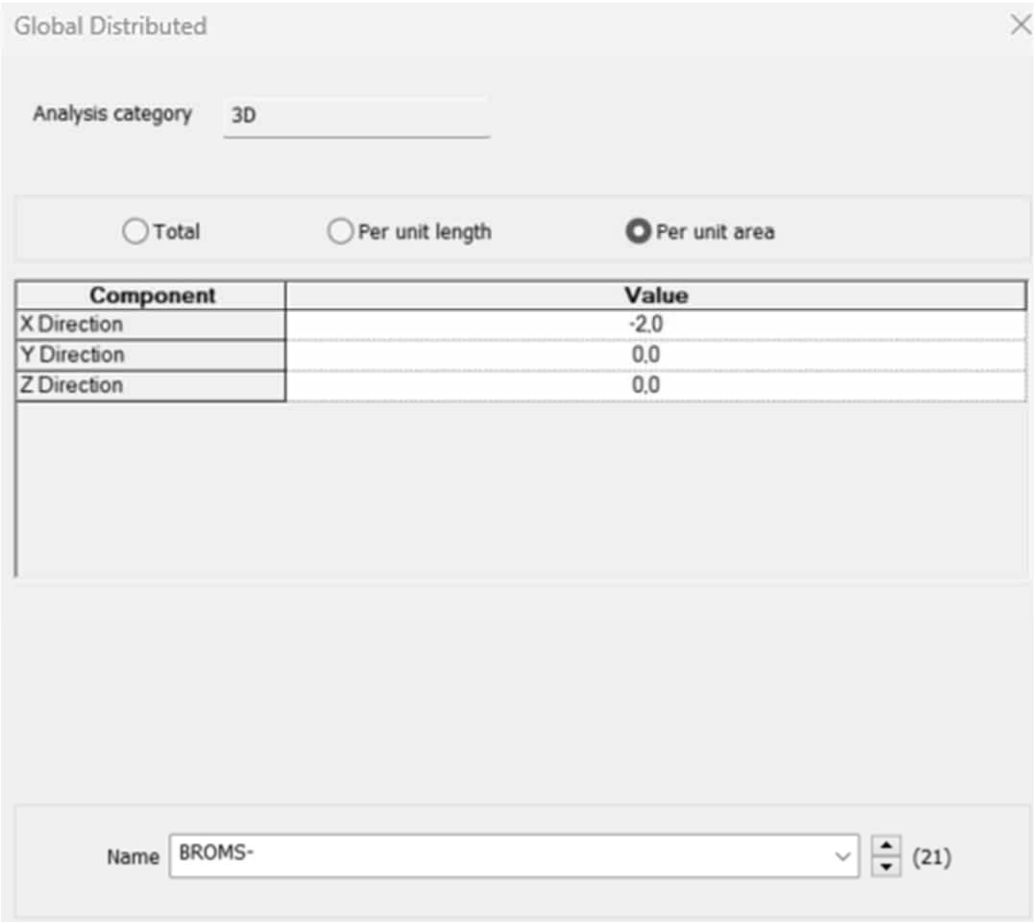
Load case : BROMS +

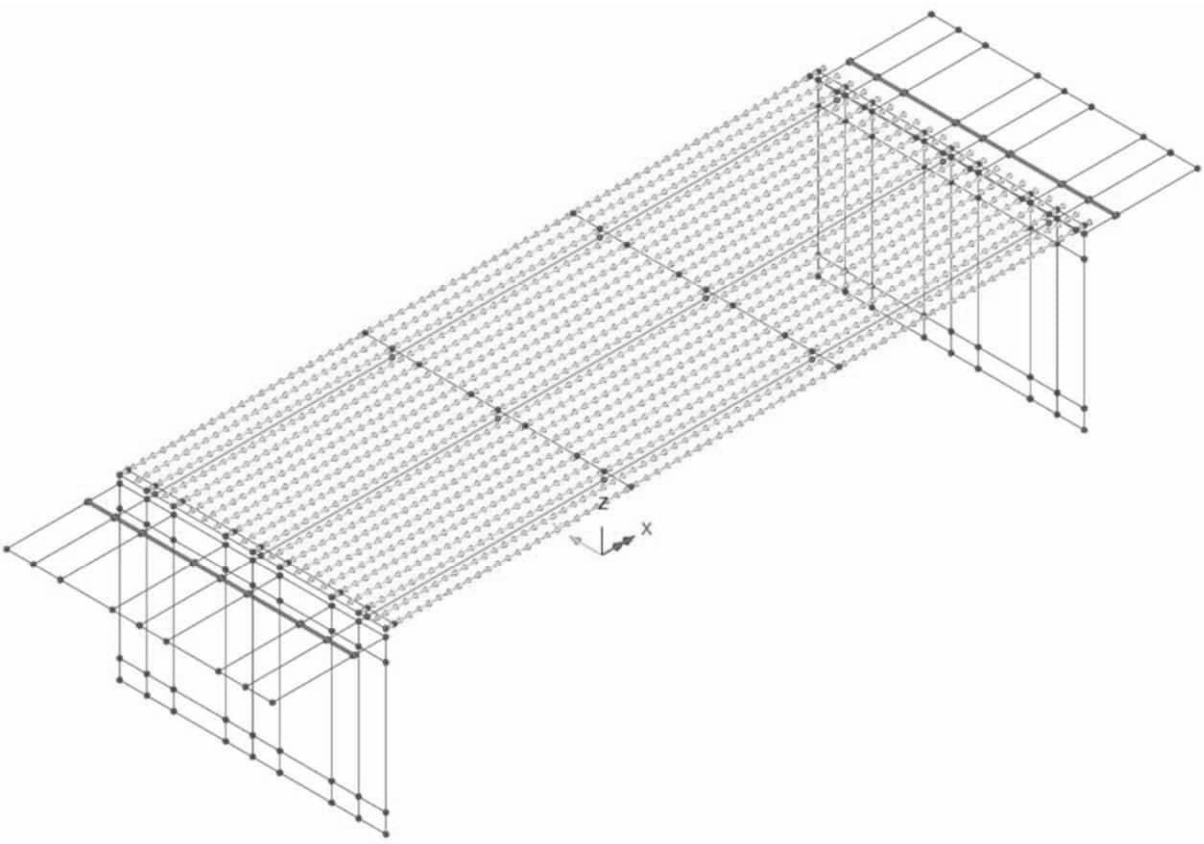




Overview 3D

Loadcase : BROMS-





Overview 3D

3.8.2 Load combination (BROMS)

Envelope BROMS :

Load case
BROMS +
BROMS -

3.9 LATERAL FORCE

Load applied to Analysis : *Analysis 1*

Lateral force is defined by SS-EN 1991-2 §4.4.2.

The load is orthogonal to braking force and acts due to skewed braking.

The acting load acts at the level of the pavement and evenly distributed over the load length.

Load model LM 1 :

$Q_{tk} = 0.25 Q_{lk} = 0.25 \cdot 402 kN = 100 kN$: skewed braking

Load model EG B = 300 kN (see TSFS chapter 11 §2) :

$Q_{tk} = 0.25 Q_{lk} = 0.25 \cdot 397 kN = 69 kN$: skewed braking

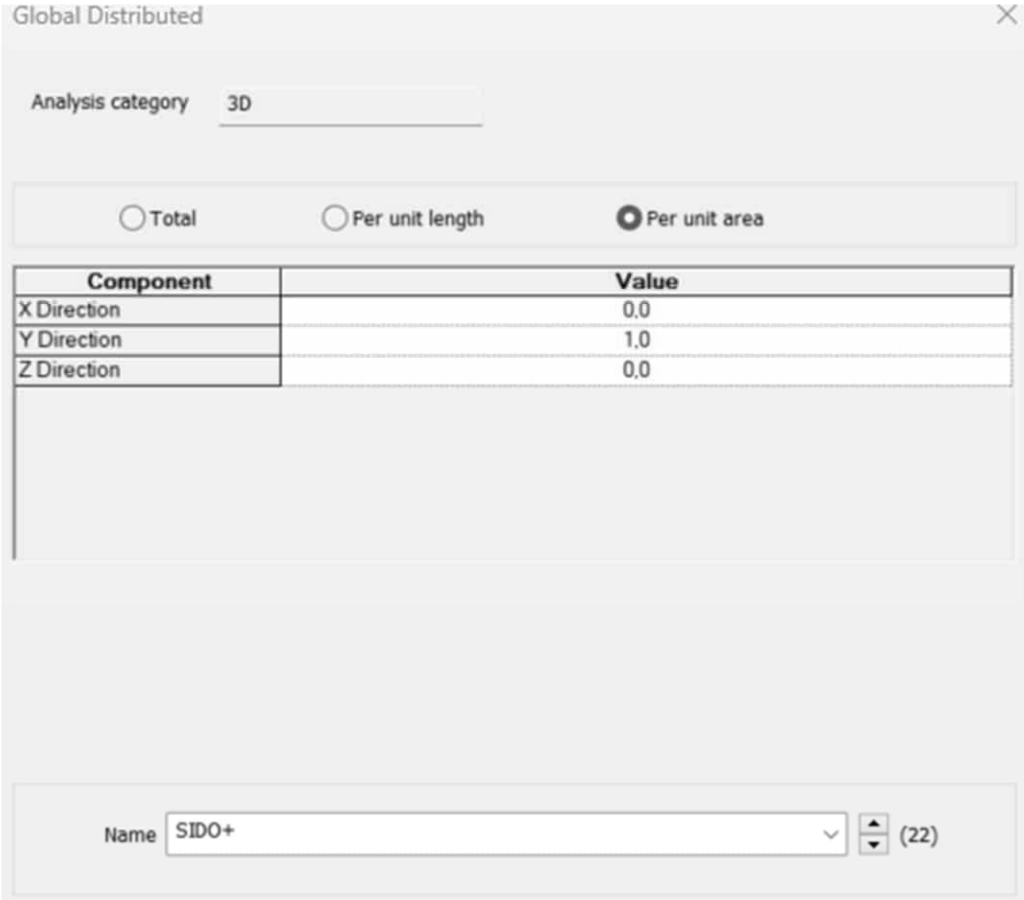
Last definition:
The load is applied as a surface load on the system line of the bridge deck, which is located 0.35 m below the pavement level. In the static model, the impact of m_x is not considered as it is deemed negligible.
To avoid considering varying load placement longitudinally, it is assumed that two lateral forces occur symmetrically on the bridge deck. This simplification of lateral forces is considered safe. The lateral force is considered evenly distributed over the entire bridge deck.

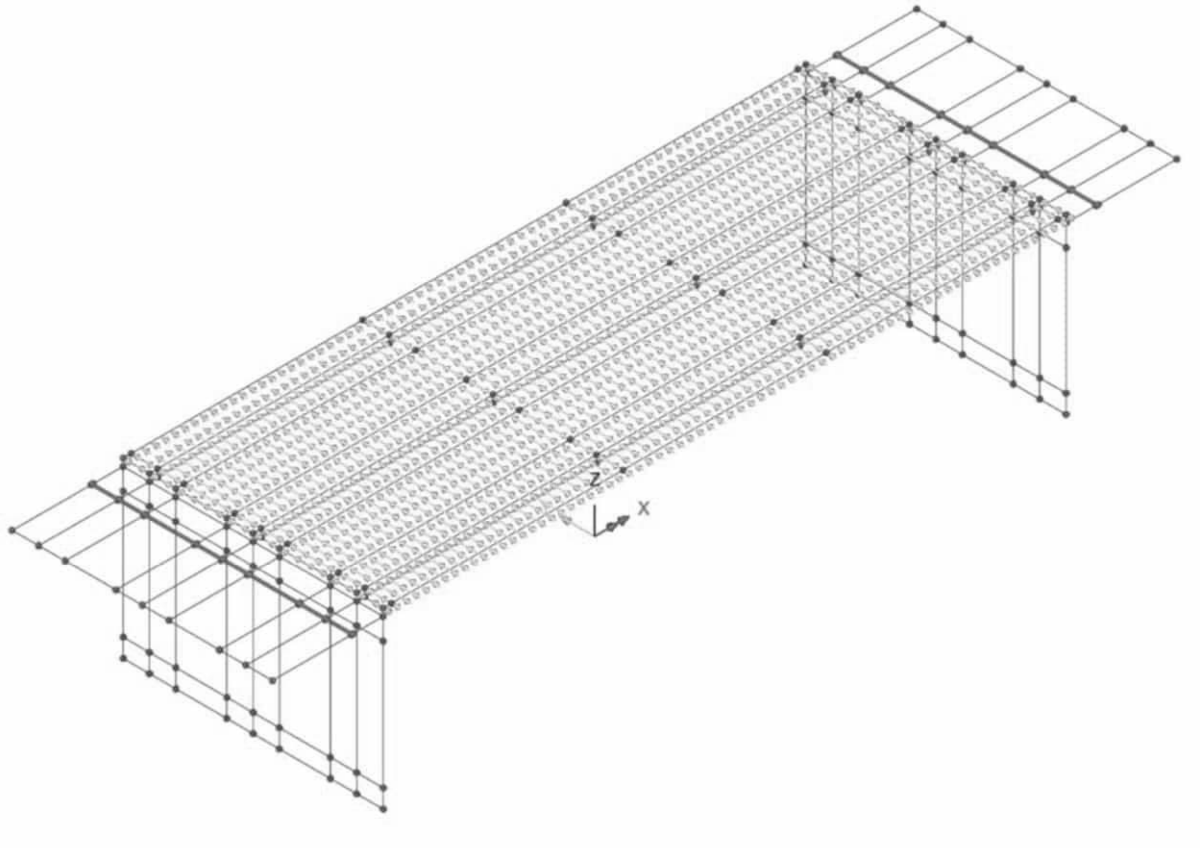
$$q_y = 2 \cdot \frac{Q_{sido}}{w_{tot} \cdot L_{deck}} = 2 \cdot \frac{100 kN}{12.8 m \cdot 33.6 m} = 0.5 \frac{kN}{m^2}$$

$$m_x = q_y \cdot (0.15 m + t_{bel}) = 0.5 \frac{kN}{m} \cdot (0.15 m + 0.10 m) = 0.1 \frac{kNm}{m^2}$$

Note:
The braking force associated with LM 1 is applied on the safe side in the system calculation.

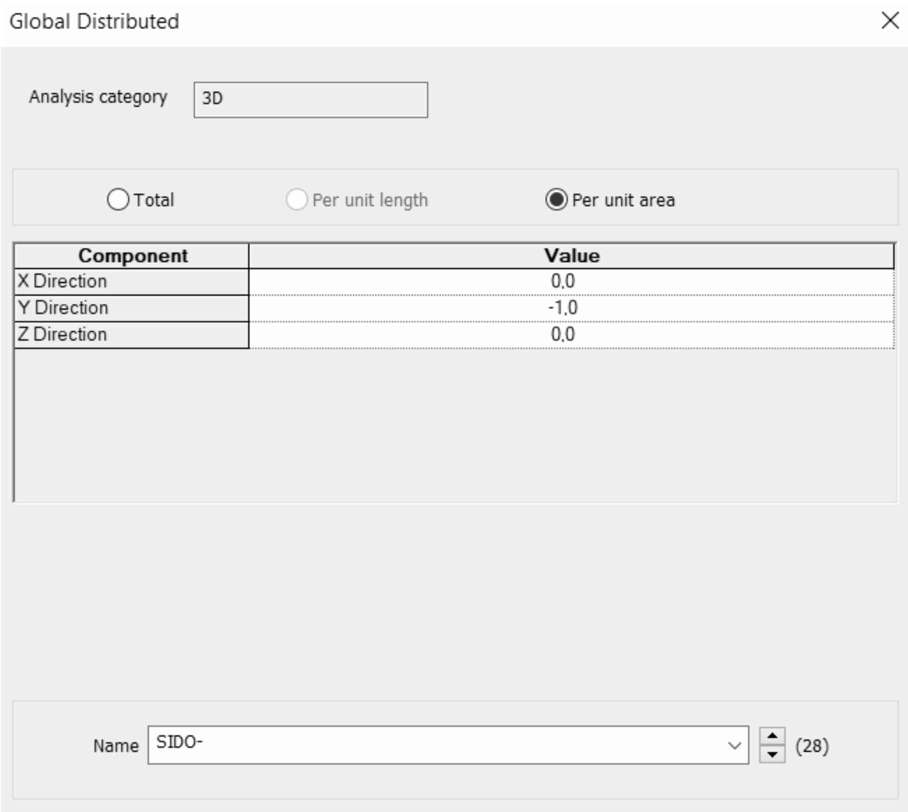
Load case : SIDO +

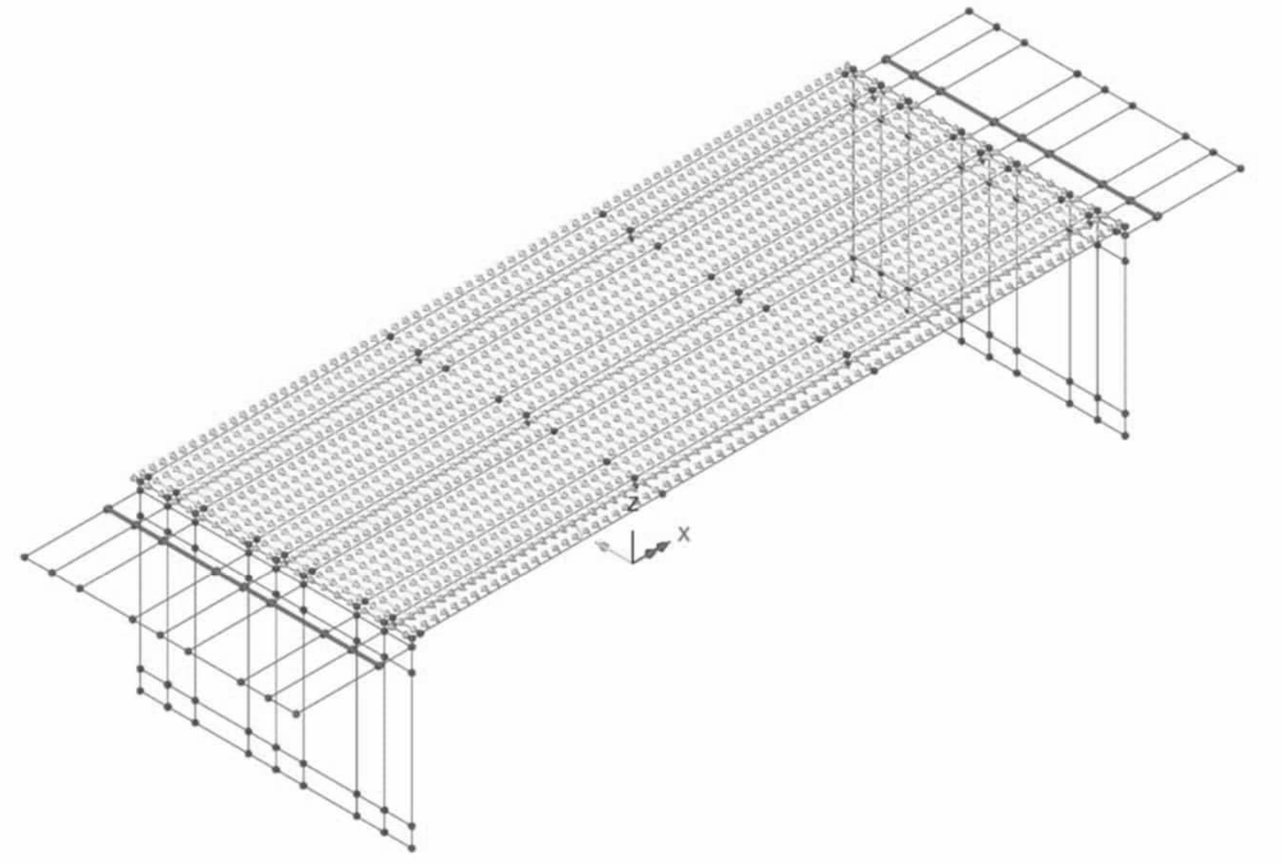




Overview 3D

Loadcase : SIDO-





Overview 3D

3.9.2.2 Load combination

Envelope SIDO :

Load case
SIDO +
SIDO -

3.10 WIND LOAD

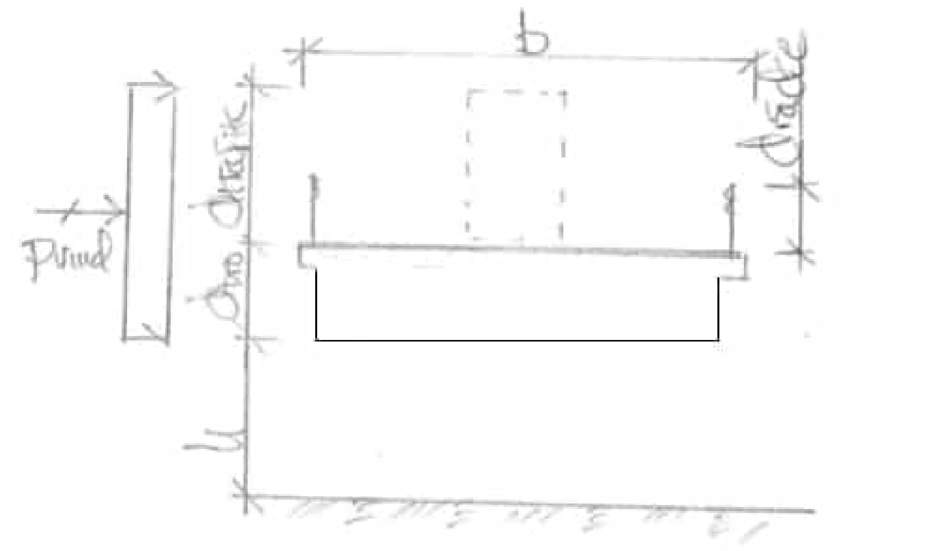
Load applied to Analysis : *Analysis 1*

Windload on bridges is defined by EN 1991-1-4 chapter 8.

Duration coefficients (see SS-EN 1990 attachment A2 table A2.1) :

$\psi_k = 1.00$
$\psi_0 = 0.30$
$\psi_1 = 0.20$
$\psi_2 = 0$

Load intensity:



Terrain type II is applied on safe side according to SS-EN 1991-1-4 table 4.1.

Reference height z_e = 6 m : SS-EN 1991-1-4 section 8.3.1 (6)

$v_b(Skellefteå; z_e = 10m; z_0 = 0.05m) = 22\frac{m}{s}$: TRVFS 2018:57 chapter 7 figure 7.1

$q_p\left(z_e = 6m, Terrängtyp\ II, v_b = 22\frac{m}{s}\right) = 0.56kPa$: TRVFS 2011:12 attachment 4 table 4.2

$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1.25\frac{kg}{m^3} \cdot \left(22\frac{m}{s}\right)^2 = 0.30\frac{kN}{m^2}$: SS-EN 1991-1-4 chapter 4.5

$c_e = \frac{q_p}{q_b} = \frac{0.56kPa}{0.30kPa} = 1.87$: SS-EN 1991-1-4 chapter 4.5

$d_{bro} = \frac{1.2m+1.7m}{2} + 0.10m = 1.55m$: construction height incl. pavement

$d_{traf} = 2.0m$: traffic height pavement

$d_{tot} = 1.55m + 2.0m = 3.55m$

$\rightarrow \frac{b_{bro}}{d_{tot}} = \frac{12.8m}{3.55m} = 3.6$

$c_{f.x0}\left(\frac{b_{bro}}{d_{tot}} = 3.6\right) = 1.4$: SS-EN 1991-1-4 sketch 8.3

$c_{f.x} = c_{f.x0} = 1.4$: SS-EN 1991-1-4 section 8.3.1 (1)

$C = c_e \cdot c_{f.x} = 1.87 \cdot 1.4 = 2.61$

Wind load structure (below pavement) :

$$\frac{A^{bro}_{ref.x}}{L} \equiv d_{bro}$$

$$p^{bro}_{vind} = \frac{F_w}{L} = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C \cdot \frac{A^{bro}_{ref.x}}{L} = \frac{1}{2} \cdot 1.25 \frac{kg}{m^3} \cdot \left(22 \frac{m}{s}\right)^2 \cdot 2.61 \cdot 1.55m = 1.2 \frac{kN}{m}$$

Wind load traffic (above pavement) :

$$\frac{A^{traf}_{ref.x}}{L} \equiv d_{traf}$$

$$p^{traf}_{vind} = \frac{F_w}{L} = \frac{1}{2} \cdot \rho \cdot v_b^2 \cdot C \cdot \frac{A^{bro}_{ref.x}}{L} = \frac{1}{2} \cdot 1.25 \frac{kg}{m^3} \cdot \left(22 \frac{m}{s}\right)^2 \cdot 2.61 \cdot 2.00m = 1.6 \frac{kN}{m}$$

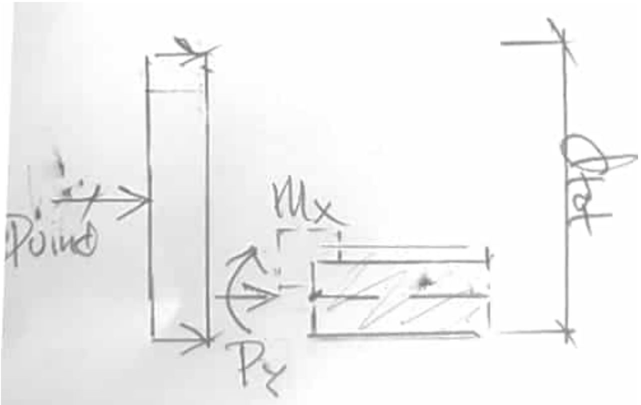
When traffic load acts om bridge together load associated to traffic can be reduced according to SS-EN 1991-1-4 section 8.1 (4). However this reduction is not considered on safe side.

$$* \, p^{traf}_{vind} = \psi_0 \cdot \frac{F_w}{L} = \psi_0 \cdot \frac{1}{2} \cdot \rho \cdot v_{b.0}^2 \cdot C \cdot \frac{A^{bro}_{ref.x}}{L} = 0.3 \cdot \frac{1}{2} \cdot 1.25 \frac{kg}{m^3} \cdot \left(23 \frac{m}{s}\right)^2 \cdot 2.61 \cdot 2.00m = 0.5 \frac{kN}{m}$$

3.10.1 Definition of load

Load is applied as a line load acting along each edge beam.

$p_{vind} = p_{vind}^{bro} + * p_{vind}^{traf} = 1.2 \frac{kN}{m} + 1.6 \frac{kN}{m} = 2.8 \frac{kN}{m}$



$\rightarrow$

$$p_y = 3 \frac{kN}{m}$$

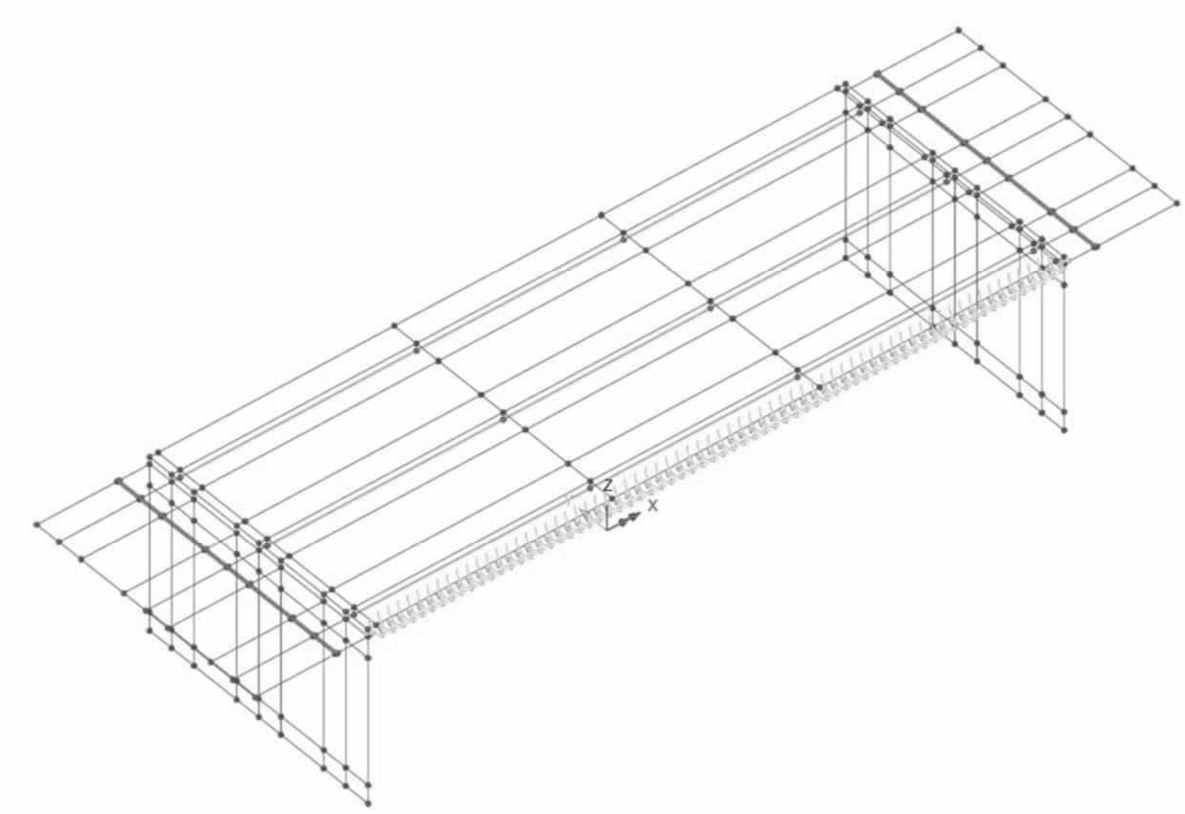
$$m_x = -p_y \cdot \left[\frac{d_{tot}}{2} - (1.45m - 0.50m)\right] = -3 \frac{kN}{m} \cdot \left[\frac{3.55m}{2} - 0.95m\right] = -3 \frac{kNm}{m}$$

Load : VIND+

Structural loading : Global distributed

Line load in Y direction (p_y) : +3 $\frac{kN}{m}$

Line moment about X axis (m_x) : -3 $\frac{kNm}{m}$

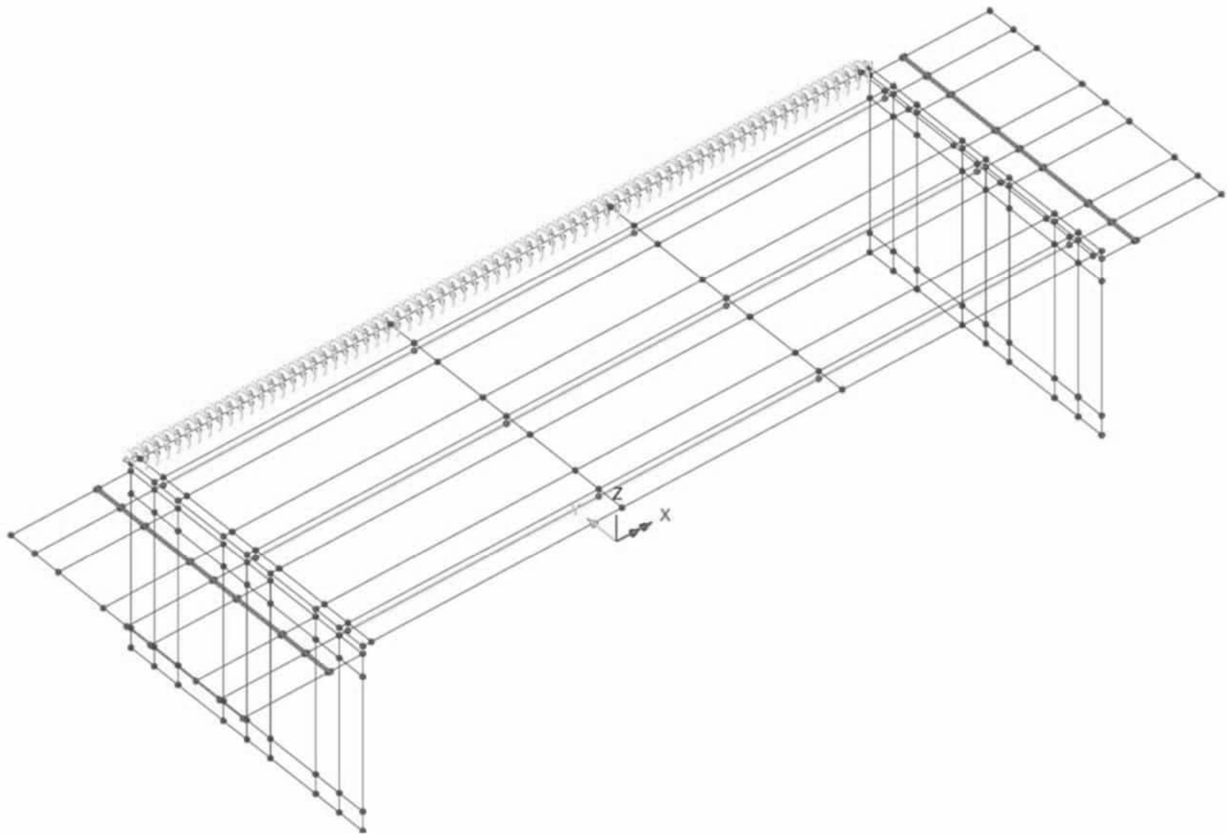


Load : VIND-

Structural loading : Global distributed

Line load in Y direction (p_y) : -3 $\frac{kN}{m}$

Line moment about X axis (m_x) : 3 $\frac{kNm}{m}$



3.10.2 Load combination

Envelope VIND :

Load case
VIND+
VIND-

3.11 SURCHARGE

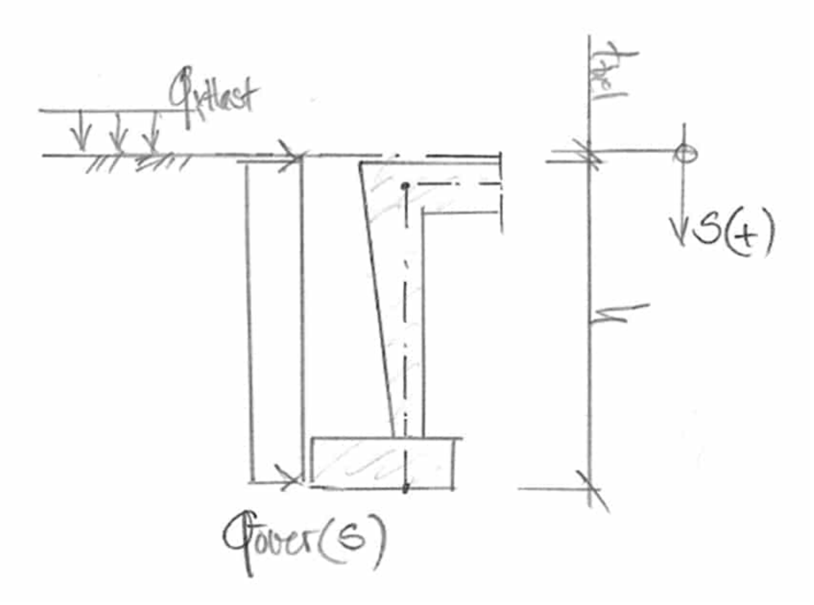
Load applied to Analysis : *Analysis 1*

TSFS chapter 11 section §8 describes load seen below.

$q_{ytlast.1} = 20kPa$: road width 6.0 m

$q_{ytlast.2} = 10kPa$: remaining width

$q_{över}\ (s) = K_0 \cdot q_{ytlast}$



$q_{ytlast}^{b=6.0m} = 0.29 \cdot 20kPa = 6kPa$

$q_{ytlast}^{övrigt} = 0.29 \cdot 10kPa = 3kPa$

Note:
Since the bridge has a link plate with a fictitious bearing located 4 meters from the back edge of the frame beam, no overload theoretically occurs towards the frame beam or wing walls. On the safe side, this favourable effect is not considered.

On the safe side, a surface load of 20 kPa is applied to the entire bridge width of 12.8 meters.

The favourable impact of the counteracting earth pressure due to movement is not accounted for on the safe side.

3.11.1 Load abutment 1

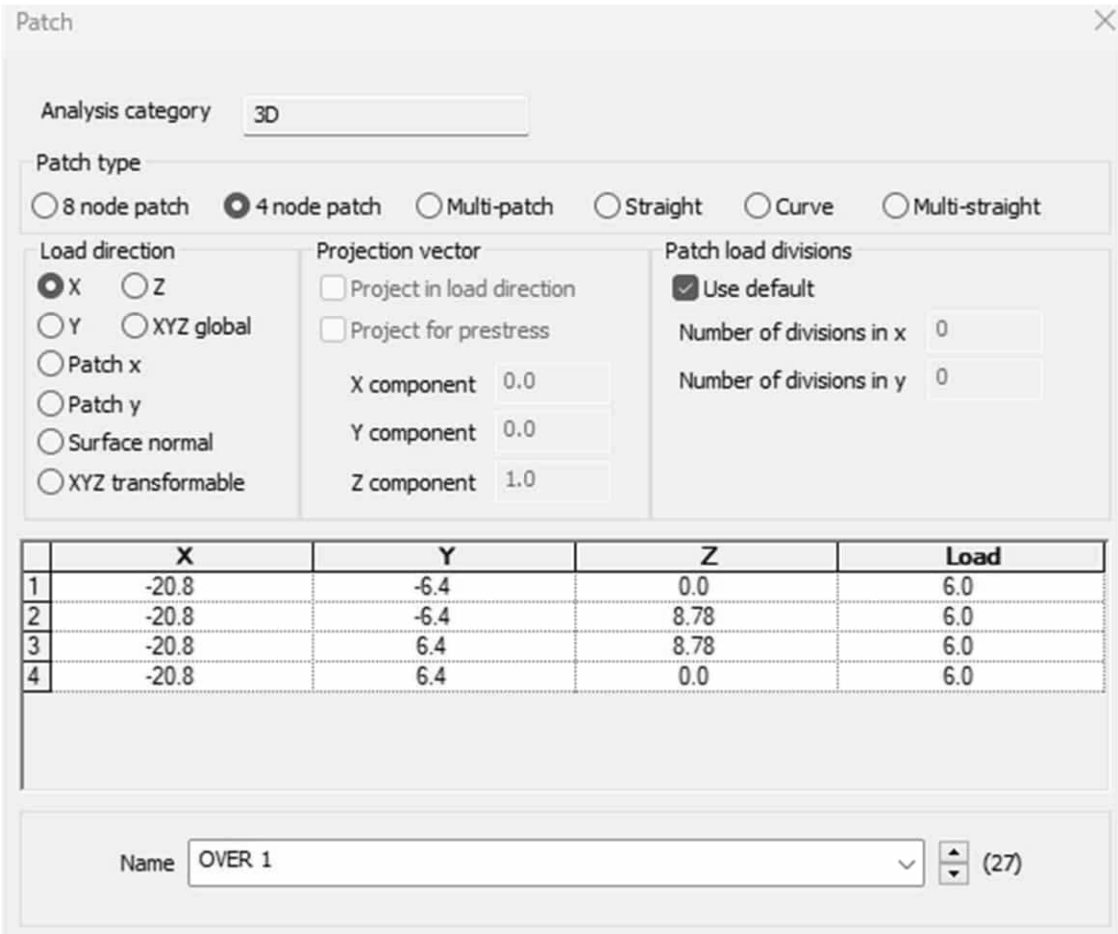
Load case : OVER 1

Structural loading : Discrete 4 node patch load

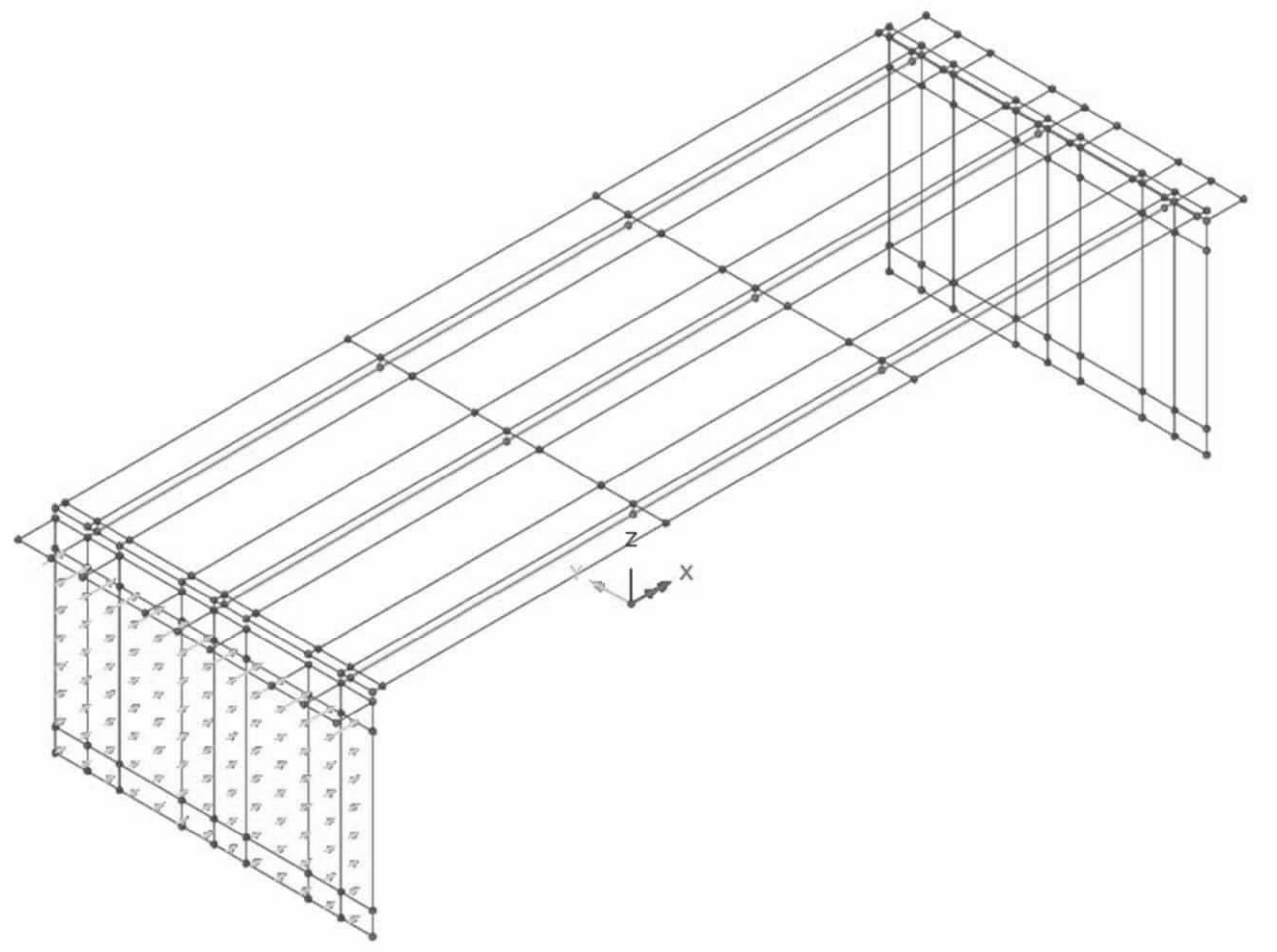
Surface load (q_x) : 6 kPa

Search Area : Abutment 1

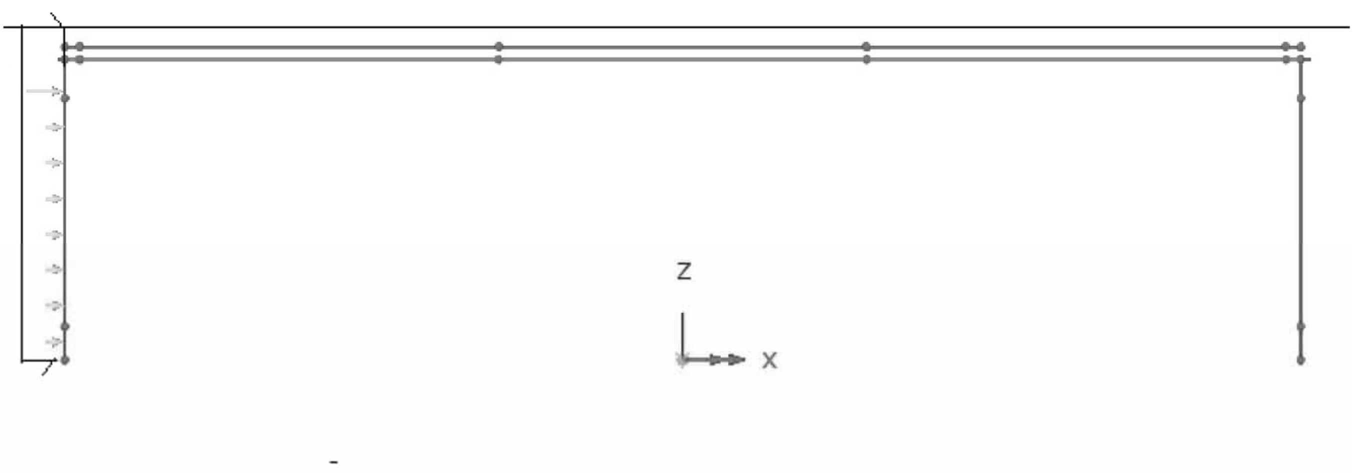
Loads outside search area : Include full load



	X	Y	Z	Load
1	-20.8	-6.4	0.0	6.0
2	-20.8	-6.4	8.78	6.0
3	-20.8	6.4	8.78	6.0
4	-20.8	6.4	0.0	6.0



Overview 3D



ELEVATION

The vector for load intensity in the figure appears to be higher at the top of the frame leg. This is because the load surface 'Abutment 1' is lower than the load surface."

3.11.2 Load abutment 2

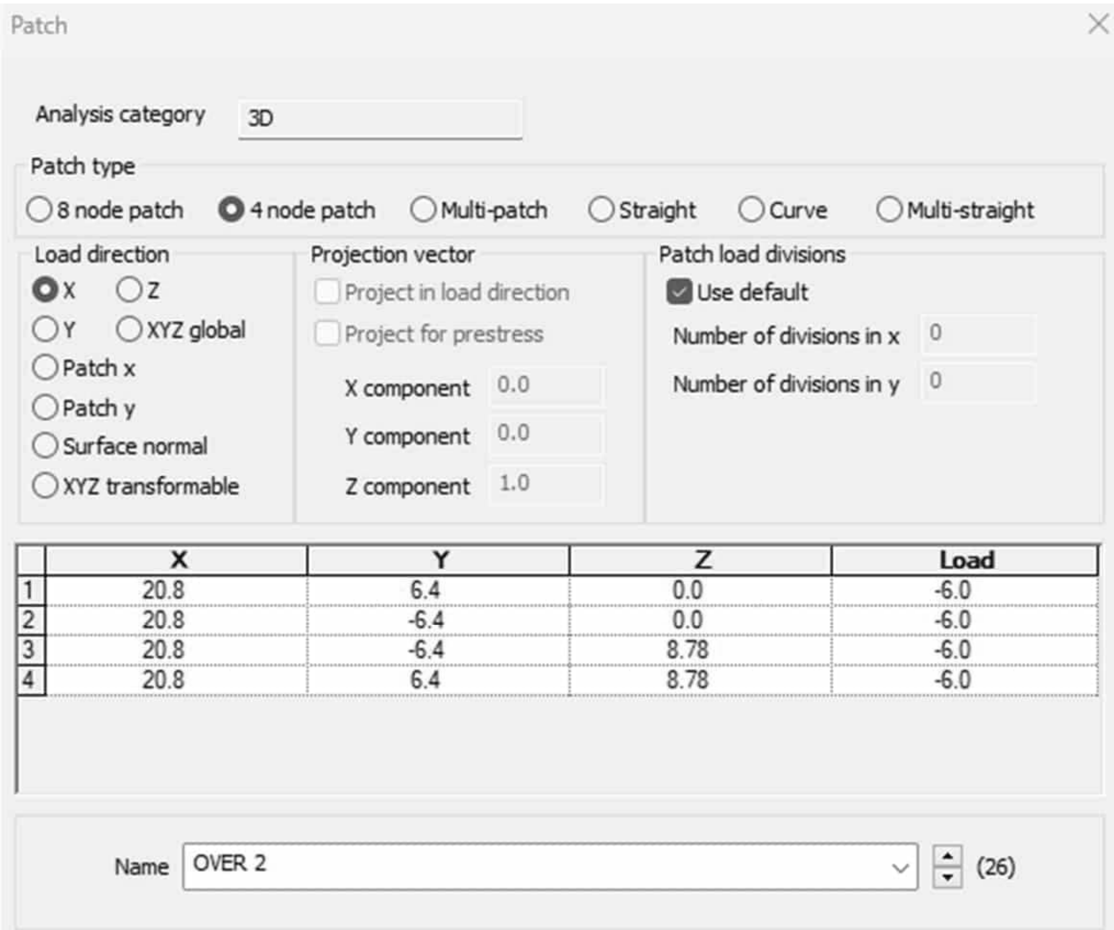
Load case : OVER 2

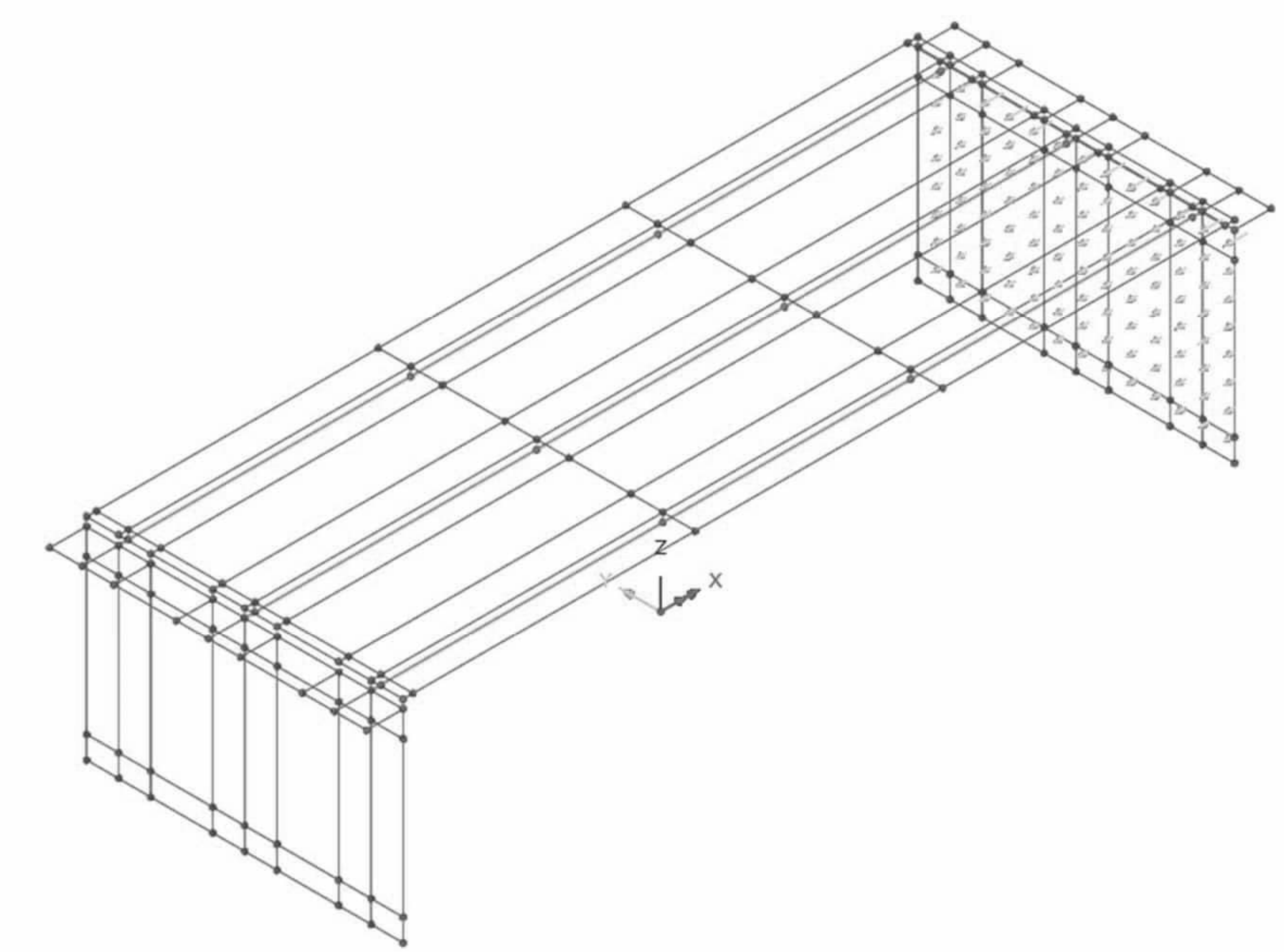
Structural loading : Discrete 4 node patch load

Surface load (q_x) : -6kPa

Search Area : Abutment 2

Loads outside search area : Include full load





Overview 3D

3.11.3 Load wingwalls

Calculation software K2.002 is used to determine the earth pressure against wing walls according to Culman's method. All wing walls are assumed to have the same length (L = 6.0 m).

Load is distributed along edge of abutments from bottom of superstructure and distance 6.08 m downward. This assumption is on safe side.

Effective height at edge abutment:

$H_{ef} = 5.0m$: see page A3:30

Forces at edge abutment in limit state (ULS):

$N_{ULS} = +133\frac{kNm}{m}$: see page A3:30

$M_{ULS} = 349\frac{kNm}{m}$: see page A3:30

Characteristic earth pressure at edge abutment:

$N_{jord} = 60\frac{kN}{m}$: see page A3:19

$M_{jord} = 154\frac{kNm}{m}$: see page A3:19

Characteristic surcharge at edge abutment:

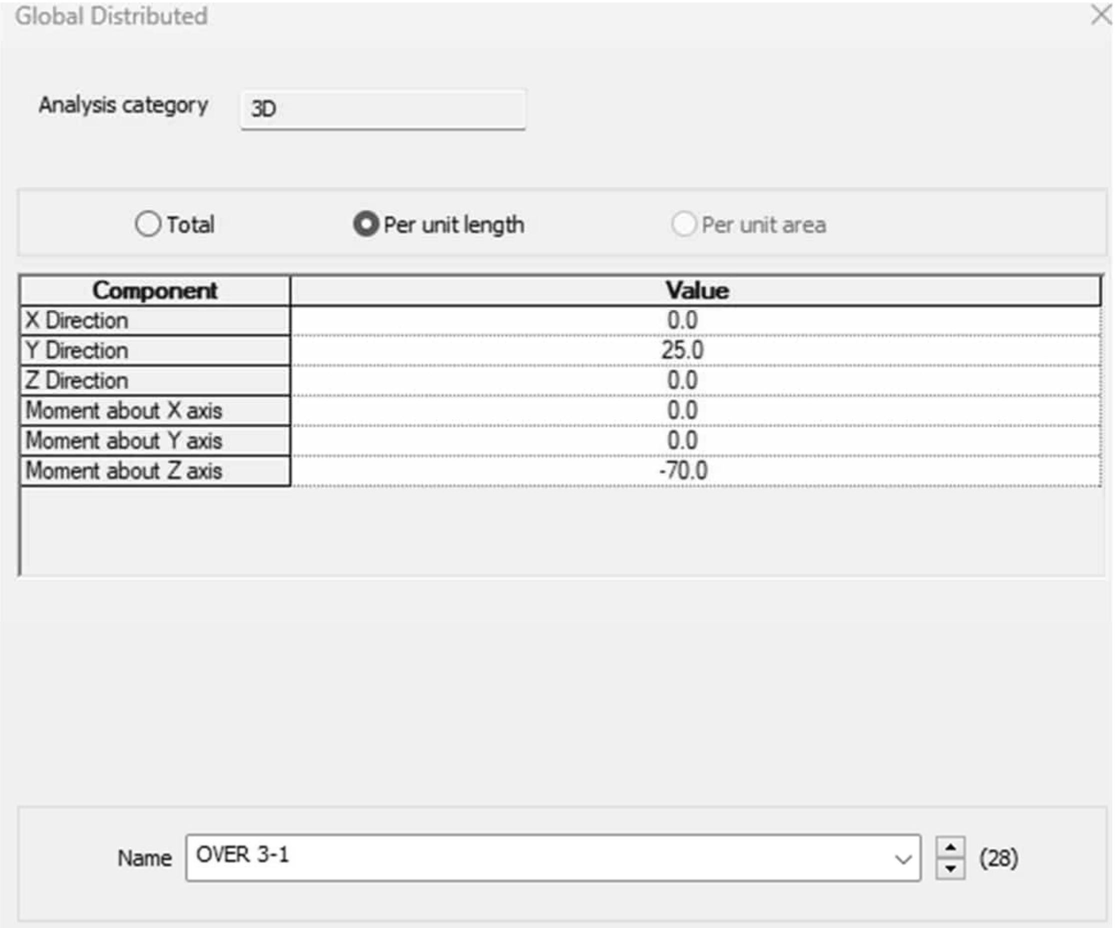
$$N_{\ddot{o}ver} = \left(133\frac{kN}{m} - 60\frac{kN}{m}\cdot 1.49\right)\cdot\frac{1}{1.71} = 25\frac{kN}{m}$$

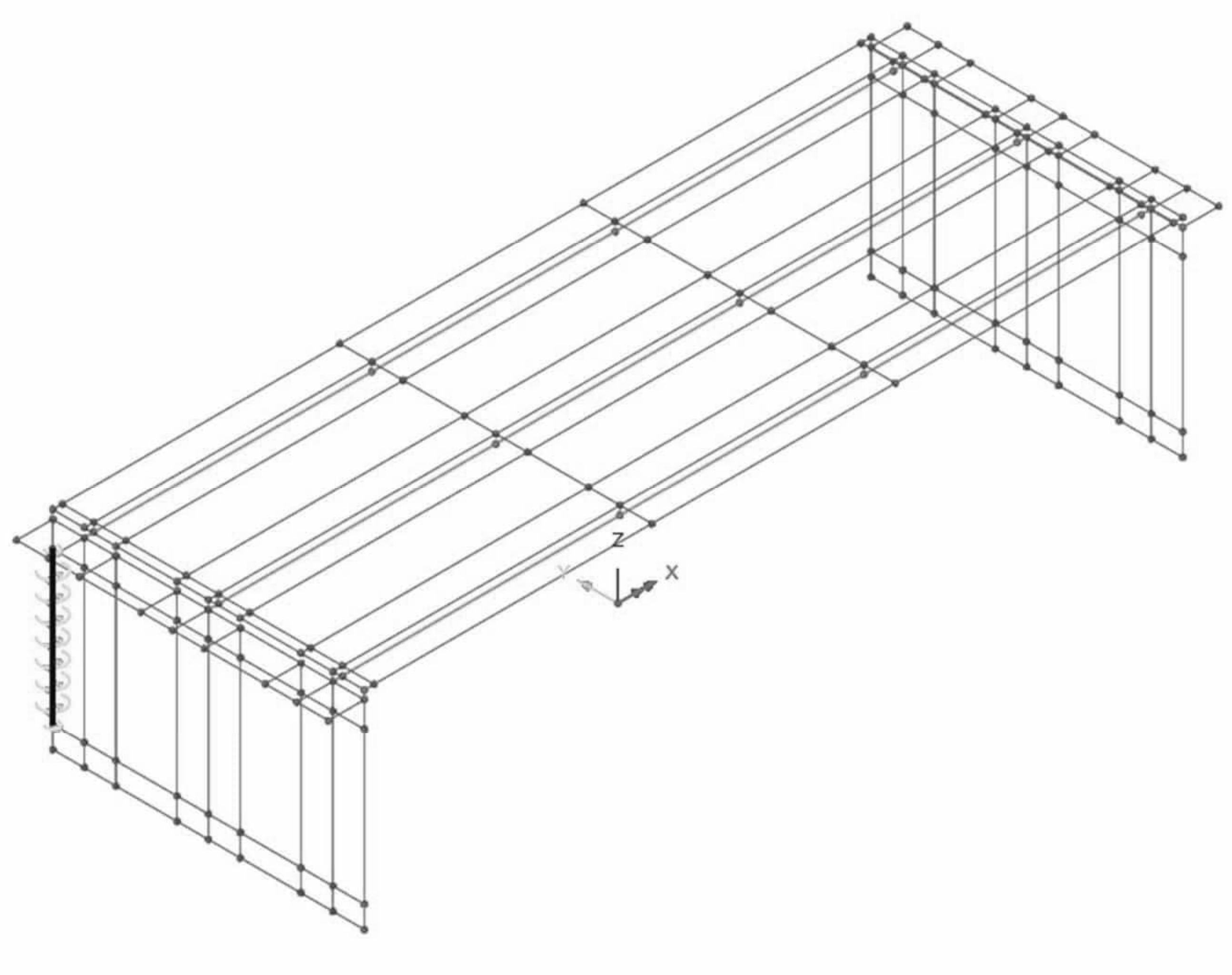
$$M_{\ddot{o}ver} = \left(349\frac{kNm}{m} - 154\frac{kNm}{m}\cdot 1.49\right)\cdot\frac{1}{1.71} = 70\frac{kNm}{m}$$

Load case : JORD 3-1
(Northern wing wall abutment 1)

$$p_y = +25\frac{kN}{m}$$

$$m_z = -70\frac{kNm}{m}$$



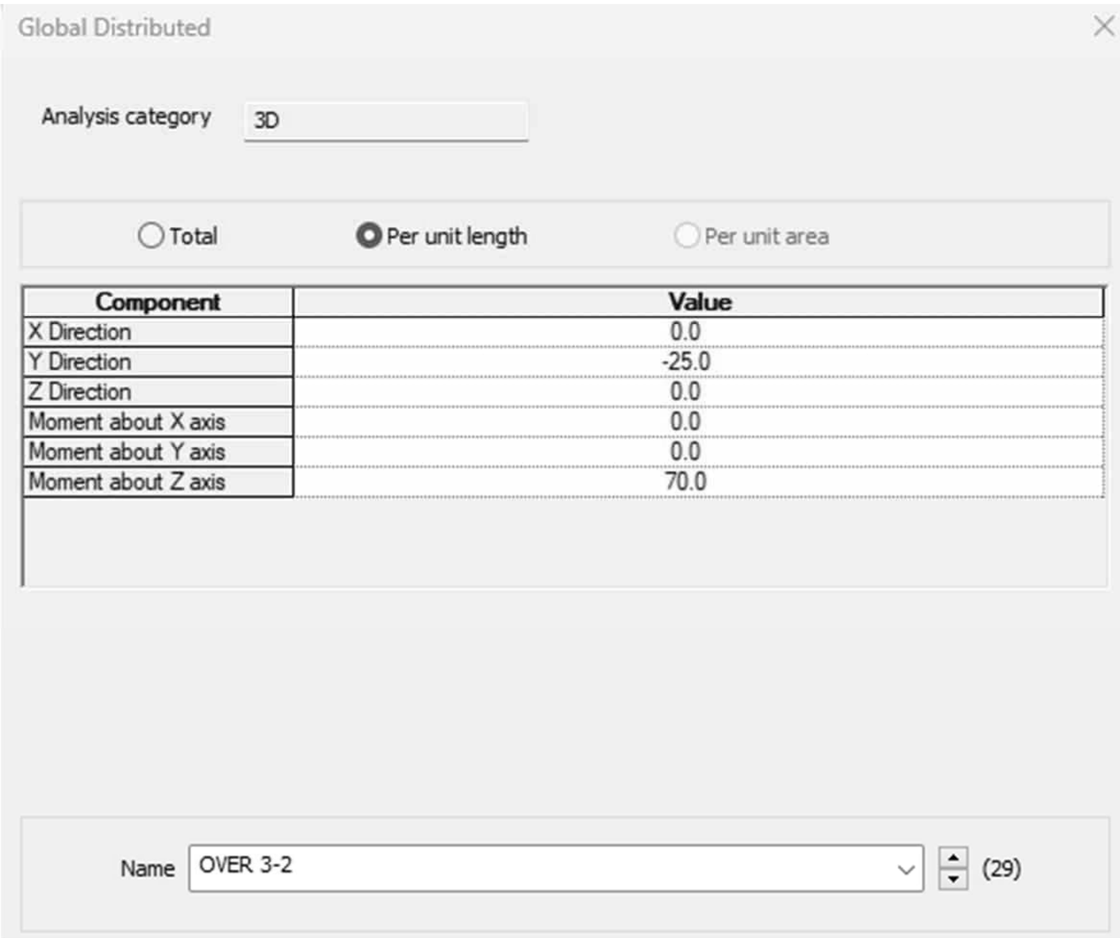


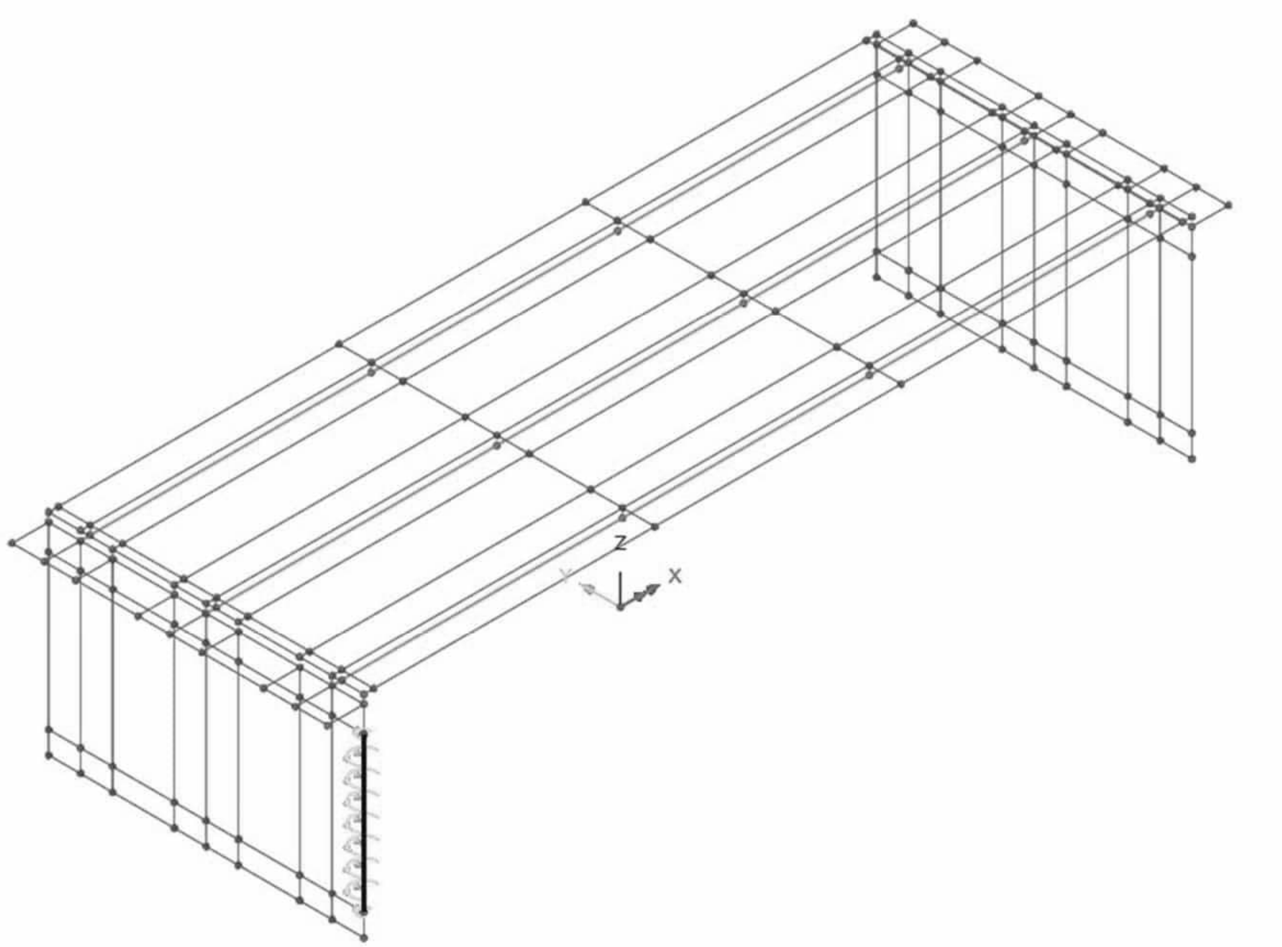
Overview 3D

Load case : OVER 3-2
(Southern wing wall abutment 1)

$$p_y = -25 \frac{kN}{m}$$

$$m_z = +70 \frac{kNm}{m}$$





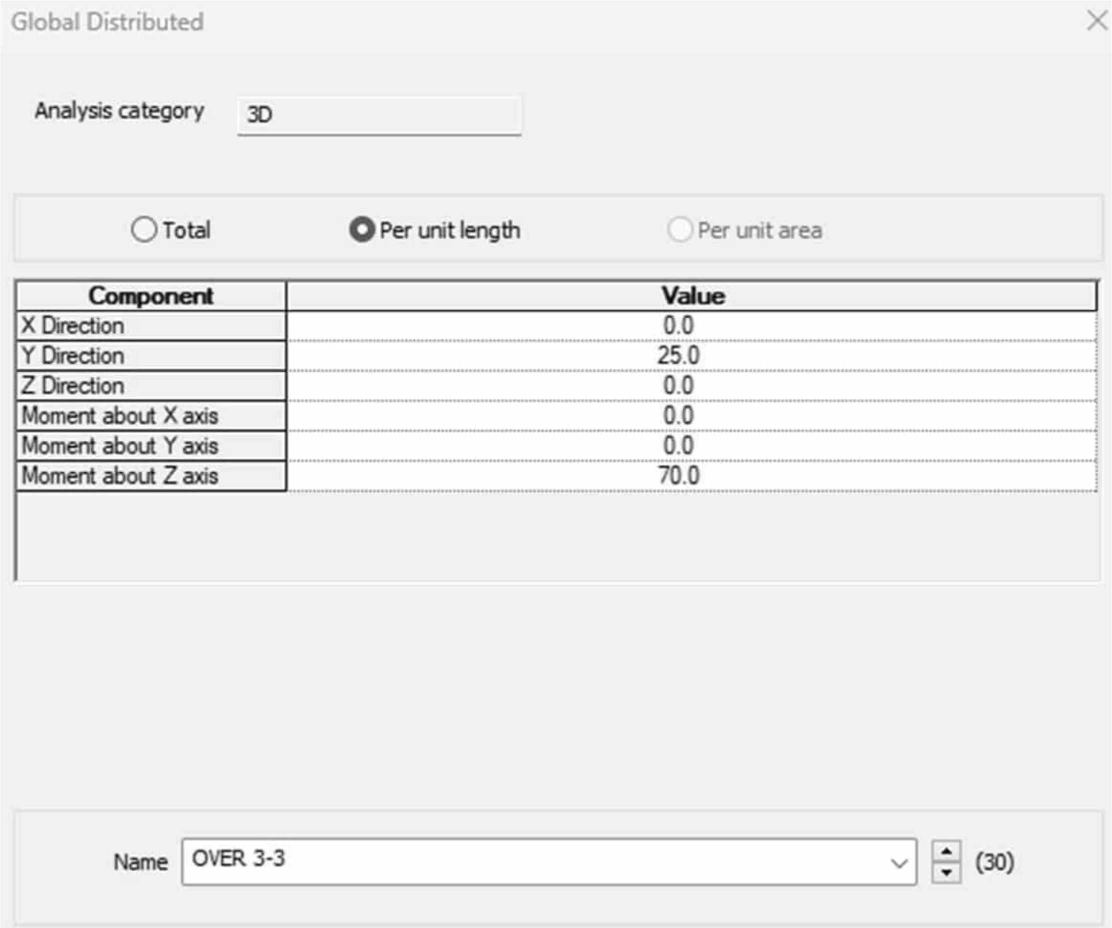
Overview 3D

Load case : OVER 3-3

(Northern wing wall abutment 2)

$p_y = +25\frac{kN}{m}$

$m_z = +70\frac{kNm}{m}$



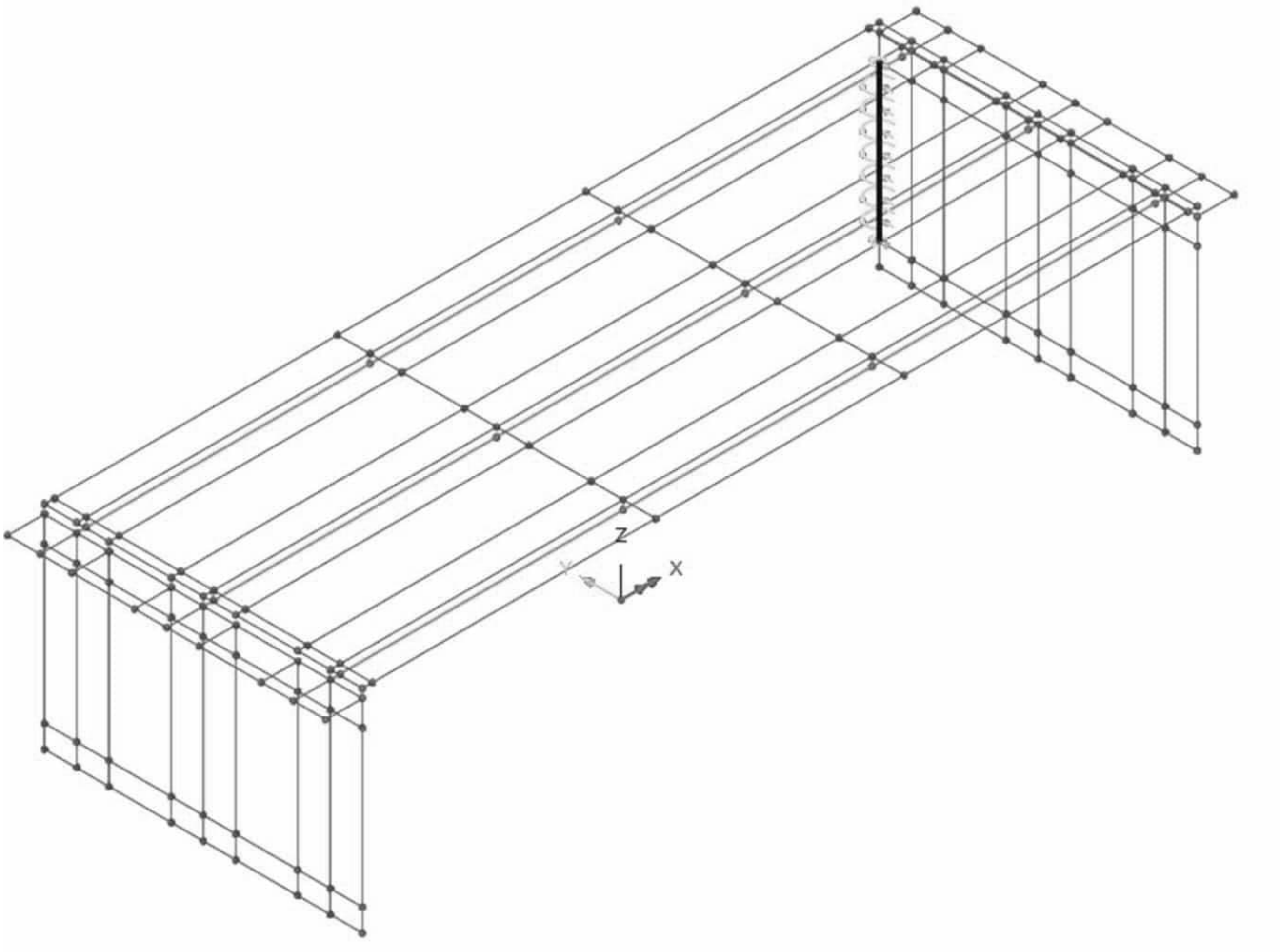
Global Distributed

Analysis category 3D

Total | Per unit length | Per unit area

Component	Value
X Direction	0.0
Y Direction	25.0
Z Direction	0.0
Moment about X axis	0.0
Moment about Y axis	0.0
Moment about Z axis	70.0

Name OVER 3-3 (30)

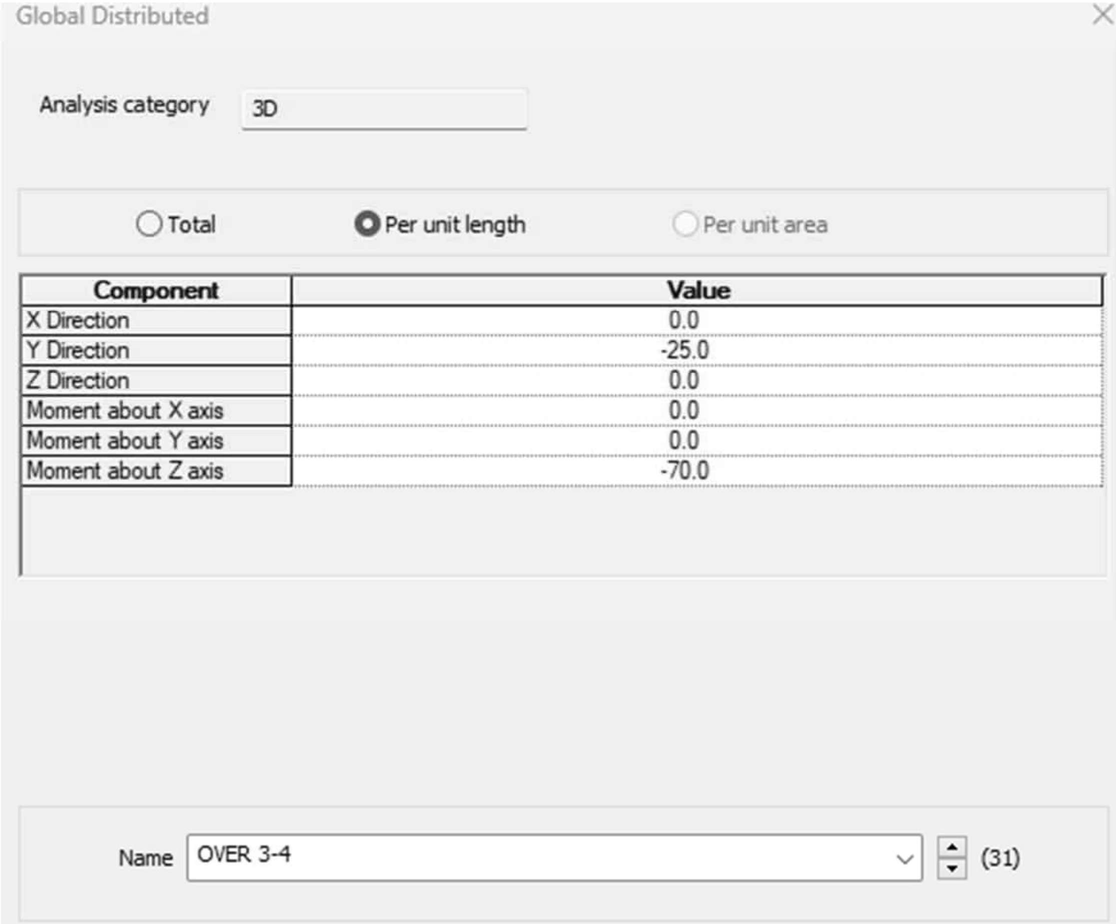


Overview 3D

Load case : OVER 3-4
(Southern wing wall abutment 2)

$$p_y = -25\frac{kN}{m}$$

$$m_z = -70\frac{kNm}{m}$$

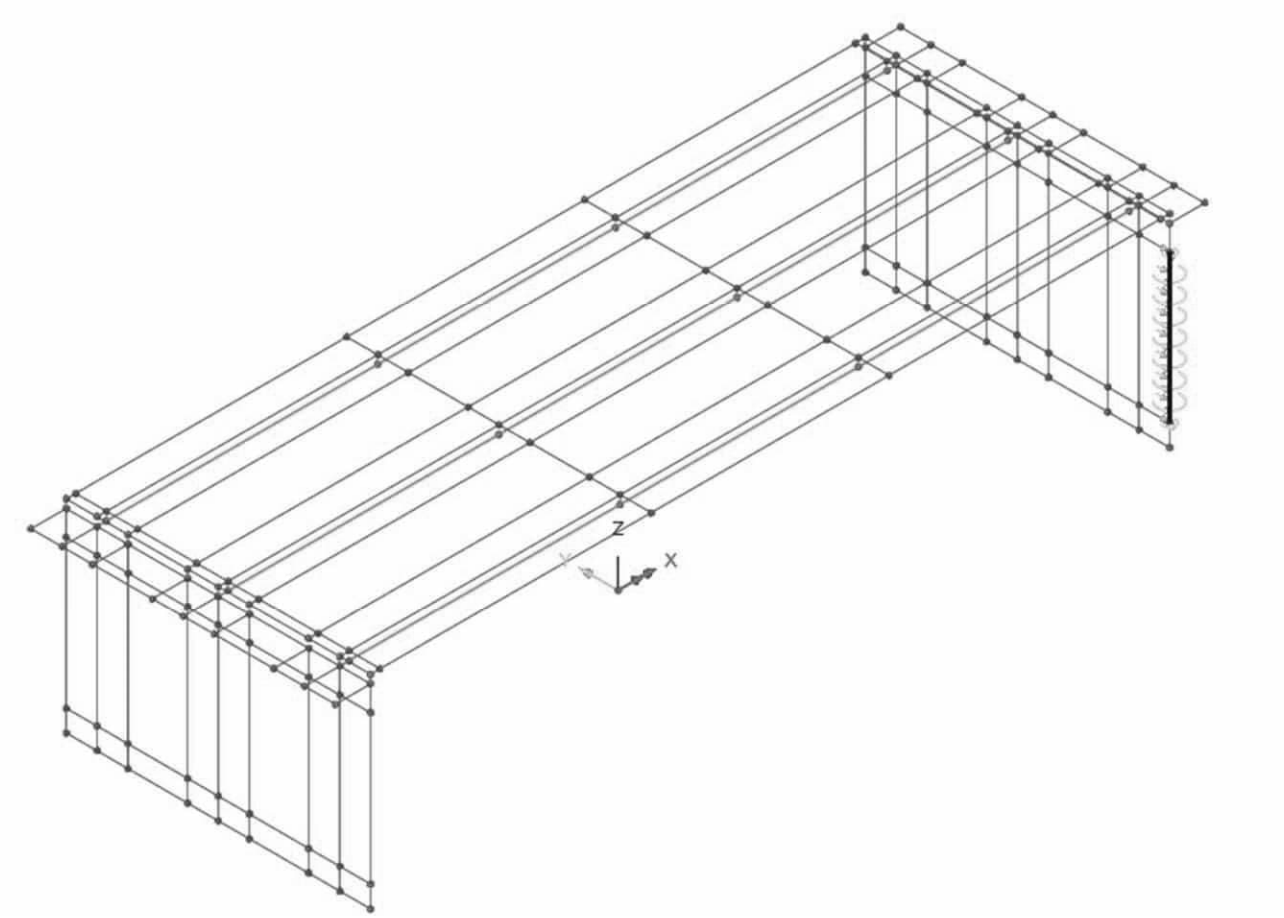

Global Distributed

Analysis category: 3D

Total / **Per unit length** (selected) / Per unit area

Component	Value
X Direction	0.0
Y Direction	-25.0
Z Direction	0.0
Moment about X axis	0.0
Moment about Y axis	0.0
Moment about Z axis	-70.0

Name: OVER 3-4 (31)



Overview 3D

3.11.4 Load combination

Load combination smart OVER :

Load case	Permanent factor	Variable factor
OVER 1	0	1
OVER 2	0	1
OVER 3-1	0	1
OVER 3-2	0	1
OVER 3-3	0	1
OVER 3-4	0	1

3.12 TEMPERATURE

Load applied to Analysis : *Analysis 2*

Temperature effect bridges according to TSFS section B.3.2.5 and EN 1991-1-5 chapter 6.

Effect in service state see SS-EN 1992-1-1 §2.3.1.2. If used then apply effect of gradual cracking according to SS-EN 1992-1-1 §5.4(3).

Effect in ultimate state is not required according to SS-EN 1992-1-1 §2.3.1.2. If used apply reduced stiffness according to SS-EN 1992-1-1 §5.4(3).

Casting temperature, T_{mont} = +10° C : EN 1991-1-5A.1(3)

Expansion coefficient, $\alpha = 12 \cdot 10^{-6}$

Concrete slab ⇒ typ 3

Location : Skellefteå

$T_{max} = +34℃$: TSFS chapter 8 sketch 8.1
$T_{min} = -42℃$: TSFS chapter 8 sketch 8.2

Duration coefficients :

Coefficients according to SS-EN 1990/A1 table A2.3

$\psi_0 = 0.60$
$\psi_1 = 0.60$
$\psi_2 = 0.50$

System superstructure:
Analysis 2 is used (see page A2:57).

13.12.1 Effect of concrete stiffness due to cracking

Impact is considered in the serviceability limit state according to SS-EN 1992-1-1 §2.3.1.2 (1). If this is done, a gradual development of cracking may be applied according to SS-EN 1992-1-1 §5.4(3).

In the studied bridge, all concrete is assumed to be cracked for the load cases of temperature, support settlement, and creep.

This assumption will be verified to ensure that it is accurate. If it is not, the calculation model will be adjusted accordingly.

Verification of whether the concrete is cracked will be done according to SS-EN 1992.2 section 7.2 (102) under the condition $\sigma_{ct}^{SLS-K} > f_{ctm}$.

A review of the bending stiffness according to SS-EN 1992-1-1 shows that completely uncracked reinforced concrete has a stiffness that is 13% of the cracked concrete according to SS-EN 1992-1-1 section 7.4.3 when applied to a slab with a thickness of 1200 mm, see page A3:116.

Selected bending stiffness in the calculation model:

In the chosen calculation, standard Swedish calculation practice is applied with stiffness of 60% for cracked compared to uncracked concrete.

Uncracked section (stage I): $EI_{osprucket} = EI_{I} = E_{cm} \cdot I_{c}$

Cracked section (stage II): $EI_{sprucket} = EI_{II} = 0.6 E_{cm} \cdot I_{c}$

Evaluation of bending stiffnesses according to SS-EN 1992-1-1 :

The control is performed for a section corresponding to b x h = 1000 mm x 1200 mm and reinforcement ϕ16s250 (∴ 804 mm^2/m).

The evaluation of stiffness for stage II (cracked section) is done using the calculation program caeEc205.

$$I_I = \frac{b \cdot h^3}{12} = \frac{1.0m \cdot (1.20m)^3}{12} = 144 \cdot 10^{-3} m^4$$: uncracked crossection

$$I_{II} = 18 \cdot 10^{-3} m^4$$: cracked crossection, see page A3: 116

$\rightarrow \eta = \frac{I_{II}}{I_I} = 13\% < 60\,\%$ thus on safe side !

```
Position: Kontroll styvhet stadium II
caeEc205-----------------------------------------------------------Version 2.2.3

Stadium I och II

Materialparametrar bruksstadie--------------------------------------------------

Betong ..........  fcm         fctk0,05     Ecm ------------------------------
                   MPa          MPa         GPa
C35/45               3,2         2,20        34,0

Materialindata------------------------------------------------------------------

Spricksäkerhetsfaktor, Zeta..........................   1,00
Effektivt kryptal, FIeff ............................   2,00
Betongens slutkrympning, ecs ........................   0,00 ‰
Elasticitetsmodul armering, Es...................... 200,00 GPa
Töjning i förespänd armering, ep ...................    0,00 ‰

Krafter + armering--------------------------------------------------------------

Moment, MEd (Positivt dragen underkant)............  1000,0 kNm
Normalkraft, NEd (Positiv draget tvärsnitt)........     0,0 kN
Normalkraftens excentricitet, e (Pos från ÖK uppåt)    0 mm
Effektiv höjd underkantsarmering, d................ 1250 mm
Armeringsarea underkantsarmering, As...............  804 mm2

Rektangulärt tvärsnitt, mått i mm-----------------------------------------------

    h      bw     bök     tök   tsök    buk    tuk   tsuk
   1200   1000      0       0      0      0      0      0

Beräkningsresultat stadium II---------------------------------------------------

Neutrallagrets läge ................................  0,174 m
Ideelt tröghetsmoment ..............................  1,818*10^-2 m4
Betongspänning tryckt kant .........................  -9,61 MPa
Stålspänning dragen kant ........................... 1043,6 MPa
```

3.12.2 Even temperature over entire bridge (JTEMP)

Even temperature over entire bridge according to EN 1991-1-5 section 6.1.3.3. This temperature change is seasonal.

Uniform temperature change across the entire bridge is given by EN 1991-1-5, section 6.1.3.3. This temperature change is seasonal and primarily causes translation from the bridge's movement center towards the respective supports. This movement is considered to give rise to increased earth pressure due to the movement.

Function according to SS EN 1991-1-5 sketch 6.1 (bridge type 3) :

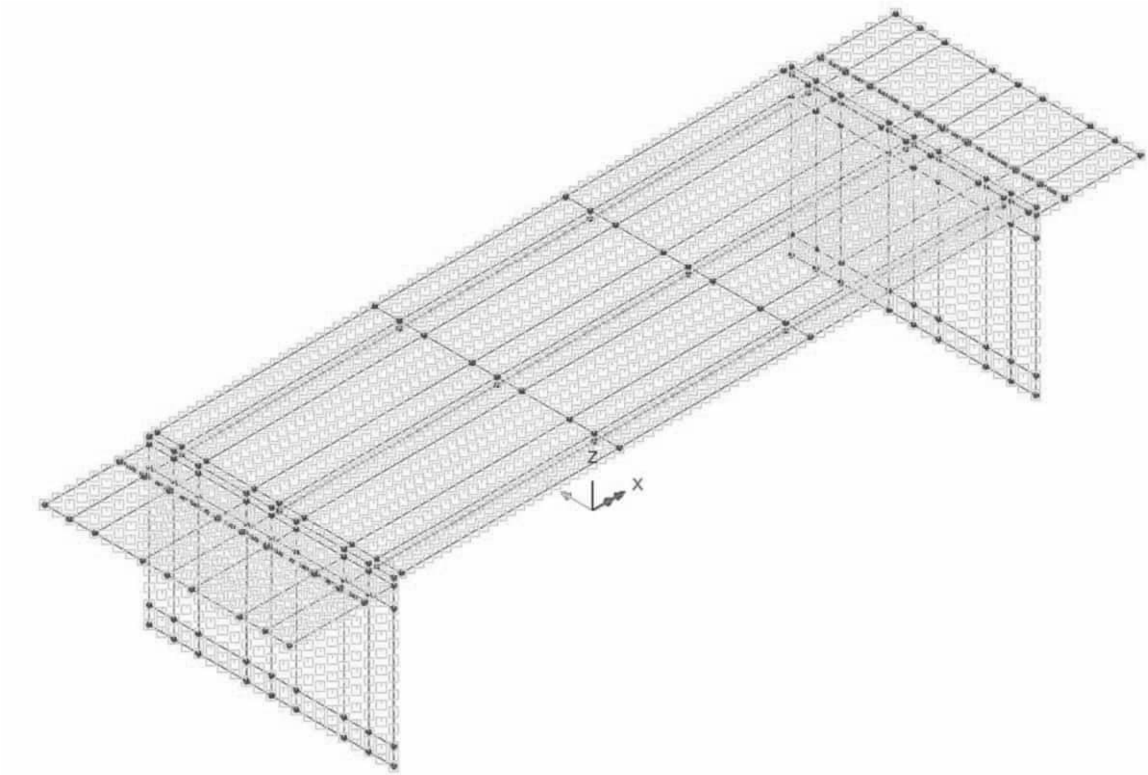
$$T_e(T) = \text{linterp}\left[(-50 \;\; 0 \;\; 30 \;\; 50)^T \cdot °C, (-42 \;\; 7 \;\; 32 \;\; 52)^T \cdot °C, T\right]$$

$$T_{e.max} = T_e\left(T_{max}\right) = 36\,°C$$

$$T_{e.min} = T_e\left(T_{min}\right) = -34\,°C$$

$$T^+ = T_{e.max} - T_0 = +36℃ - 10℃ = +26℃$$

$$T^- = T_{e.min} - T_0 = -34℃ - 10℃ = -44℃$$



Overview 3D

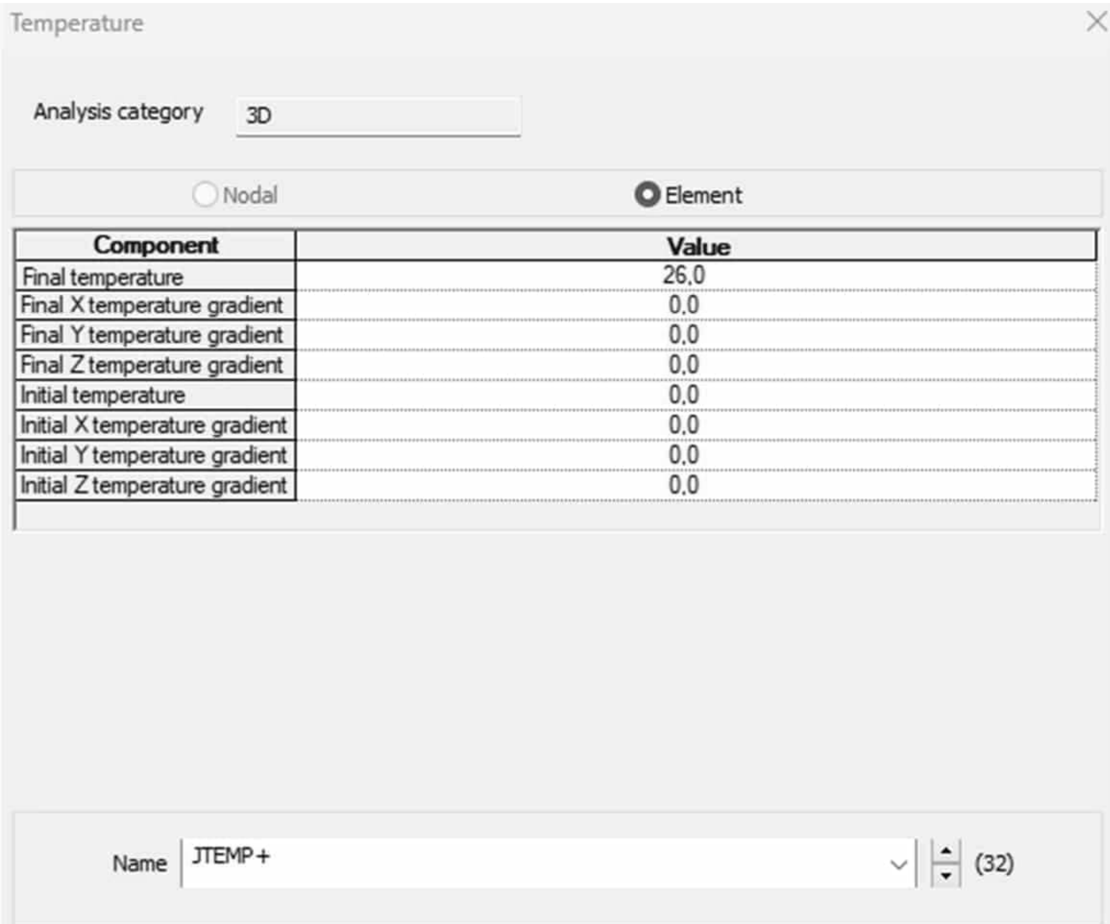
Load : JTEMP+

Structural loading : Temperature

Final temperature : +26C

Initial temperature : ±0 C

Load case : JTEMP+


Temperature

Analysis category 3D

Nodal Element

Component	Value
Final temperature	26,0
Final X temperature gradient	0,0
Final Y temperature gradient	0,0
Final Z temperature gradient	0,0
Initial temperature	0,0
Initial X temperature gradient	0,0
Initial Y temperature gradient	0,0
Initial Z temperature gradient	0,0

Name JTEMP+ (32)

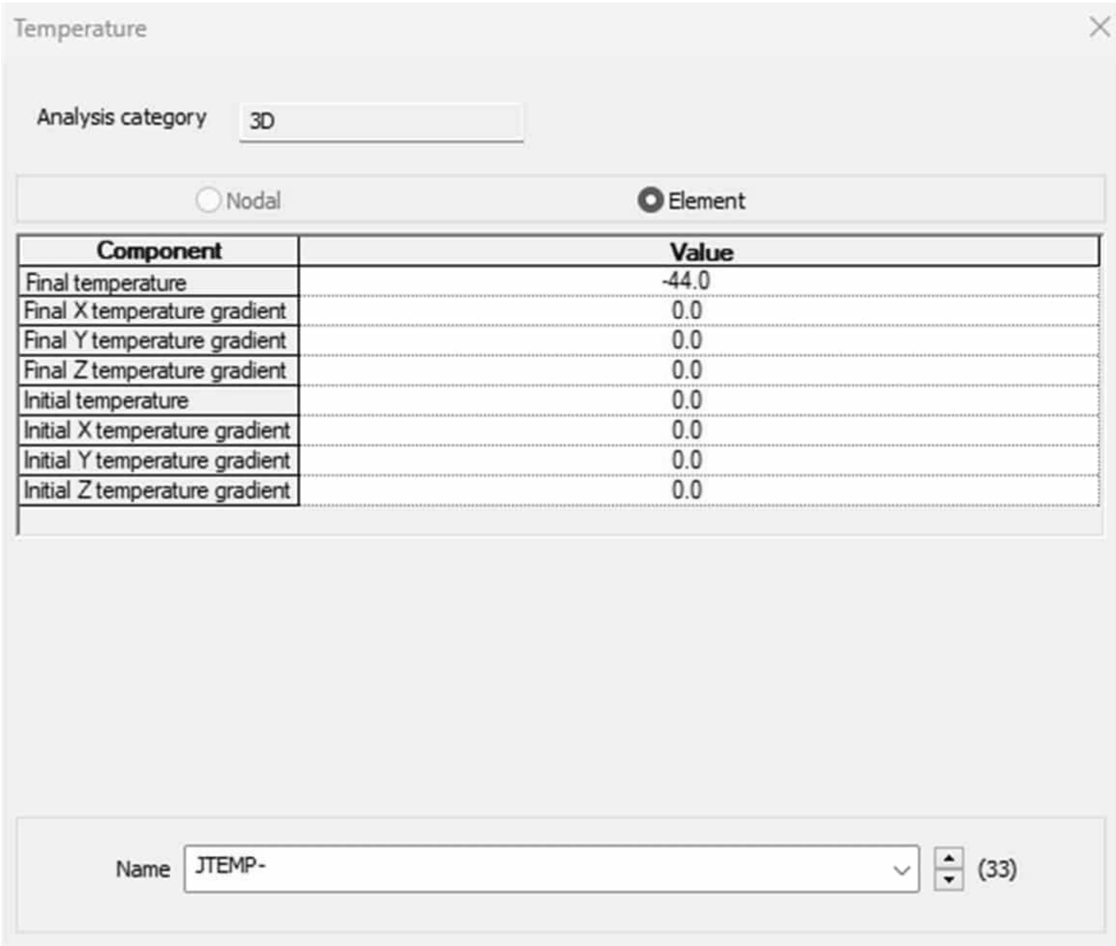
Load : JTEMP-

Structural loading : Temperature

Final temperature : -44C

Initial temperature : ±0 C

Load case : JTEMP-



Temperature

Analysis category 3D

Nodal Element

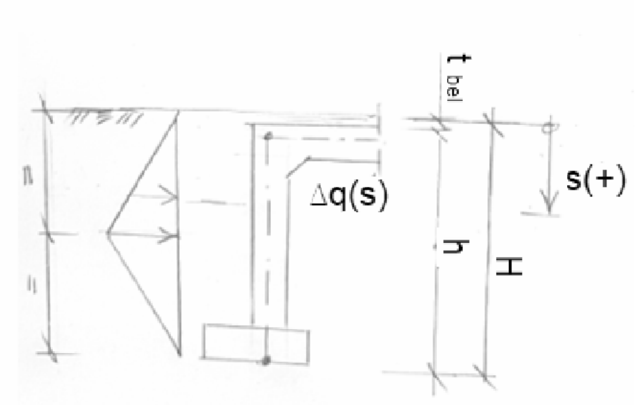
Component	Value
Final temperature	-44.0
Final X temperature gradient	0.0
Final Y temperature gradient	0.0
Final Z temperature gradient	0.0
Initial temperature	0.0
Initial X temperature gradient	0.0
Initial Y temperature gradient	0.0
Initial Z temperature gradient	0.0

Name JTEMP- (33)

3.12.2 Increased earth pressure due to movement (DELTA P)

The increased earth pressure caused by the movement of the approach slab towards the backfill is calculated according to TRVINFRA-00227, section 7.2.1..2.1. This load corresponds to what is stated in SS-EN 1997-1, section C.3. This section is used to ensure that movements for Δq(s) do not exceed the limit for passive earth pressure. During this check, "firm ground" and "wall movement type" b are applied. . Vid denna kontroll tillämpas ”fast jord” och ”väggens rörelsesätt” typ b.

$$\rightarrow \delta = (T^+ - T^-) \cdot \alpha \cdot \frac{L_{bro}}{2} = (26℃ + 44℃) \cdot 1.2 \cdot 10^{-5} \cdot \frac{36300mm}{2} = 15mm$$



$h = 8685mm$

$t_{bel} = 95mm$

$\rightarrow H = 8780mm$

$$\Delta q(s) = c \cdot \gamma \cdot s \cdot \frac{\delta}{H} = 600 \cdot 20 \frac{kN}{m^3} \cdot s \cdot \frac{15mm}{8780mm}$$

$$\Delta q_{max} = 600 \cdot 20 \frac{kN}{m^3} \cdot \frac{8780m}{2} \cdot \frac{15mm}{8780mm} = 90kPa$$

Note:
No reduction is made considering creep or cracking, as this is not an internal constraint load. This applies to both ultimate limit state (ULS) and serviceability limit state (SLS).

3.12.2.1 Load abutment 1

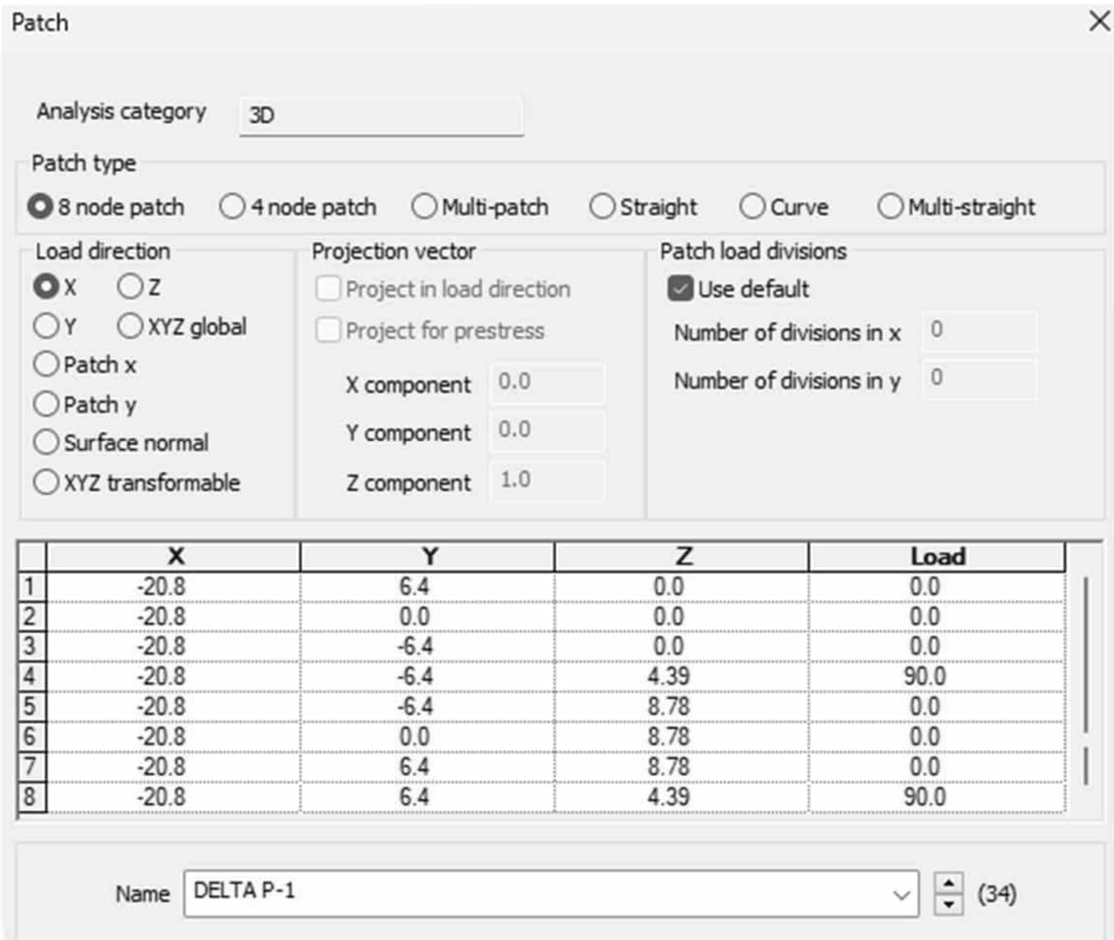
Discrete patch load : DELTA P-1

Structural loading : Discrete 8 node patch

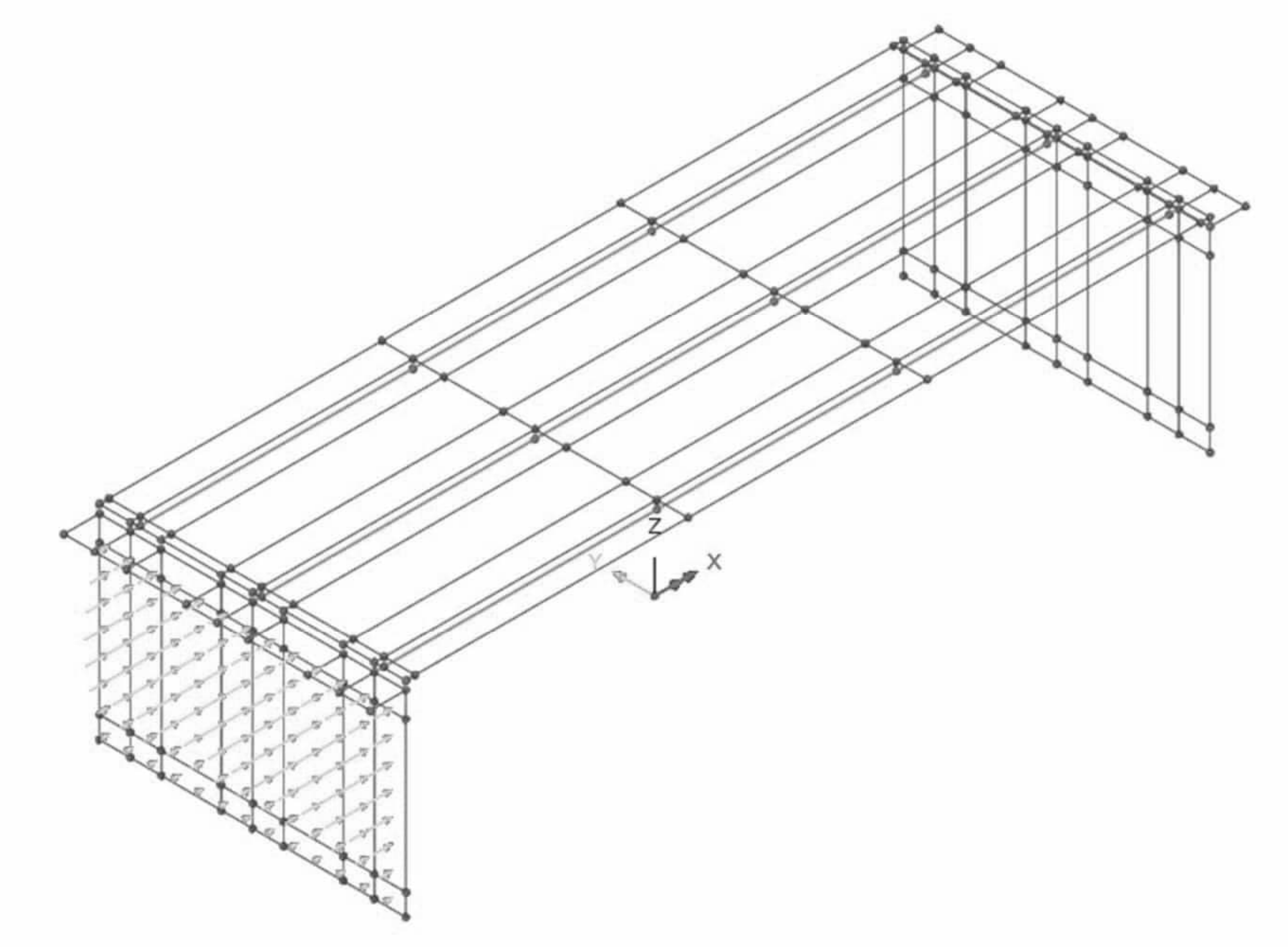
Surface load (q_x) : 0 kPa → +90 kPa

Search Area : Abutment 1

Loads outside search area : Include full load



	X	Y	Z	Load
1	-20.8	6.4	0.0	0.0
2	-20.8	0.0	0.0	0.0
3	-20.8	-6.4	0.0	0.0
4	-20.8	-6.4	4.39	90.0
5	-20.8	-6.4	8.78	0.0
6	-20.8	0.0	8.78	0.0
7	-20.8	6.4	8.78	0.0
8	-20.8	6.4	4.39	90.0



Overview 3D

3.12.2.2 Load abutment 2

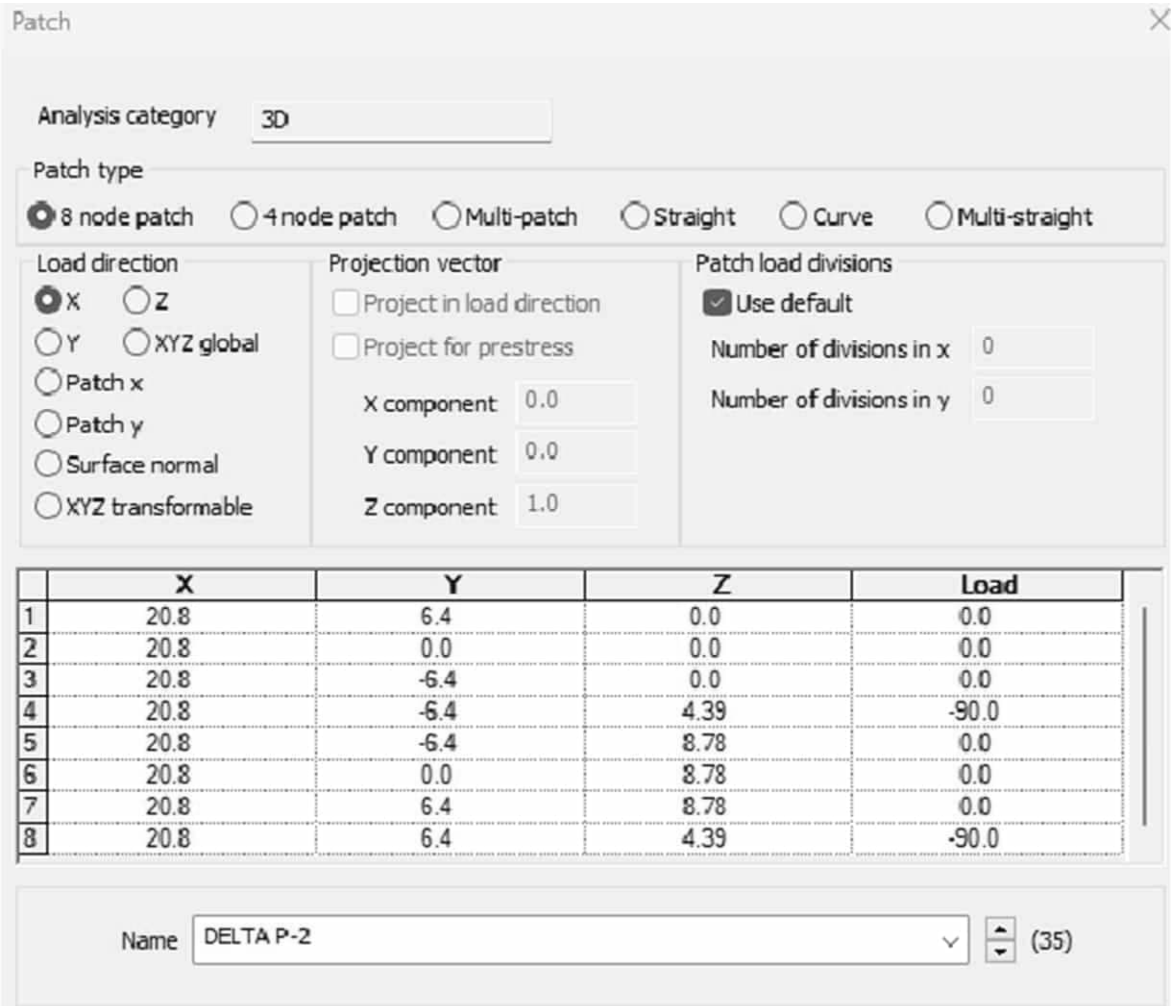
Discrete patch load : DELTA P-2

Structural loading : Discrete 8 node patch

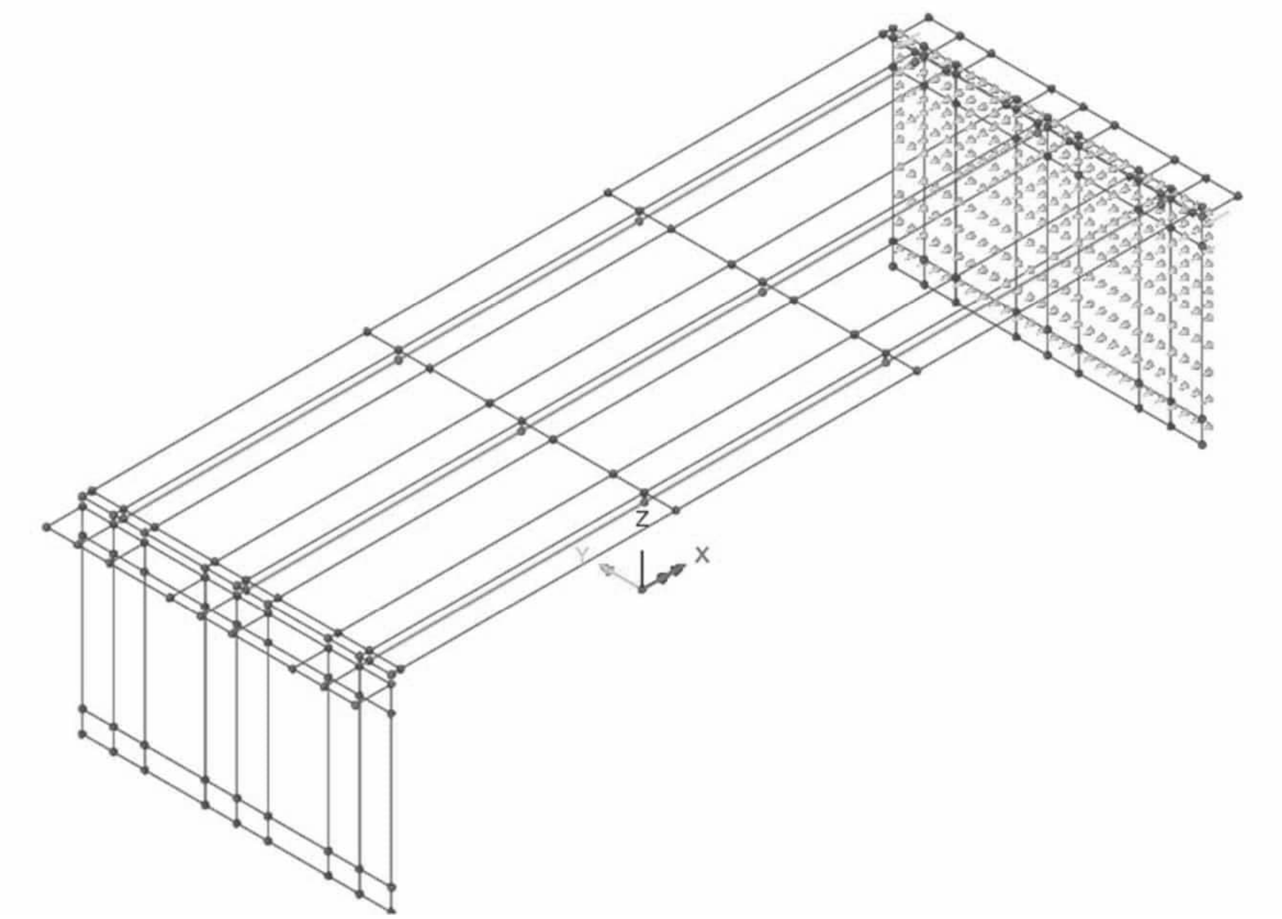
Surface load (q_x) : 0 kPa → -90 kPa

Search Area : Abutment 2

Loads outside search area : Include full load



	X	Y	Z	Load
1	20.8	6.4	0.0	0.0
2	20.8	0.0	0.0	0.0
3	20.8	-6.4	0.0	0.0
4	20.8	-6.4	4.39	-90.0
5	20.8	-6.4	8.78	0.0
6	20.8	0.0	8.78	0.0
7	20.8	6.4	8.78	0.0
8	20.8	6.4	4.39	-90.0



Overview 3D

3.12.2.3 Load combination

Load combination basic DELTA P :

Load case	Factor
DELTA P-1	1
DELTA P-2	1

3.12.3 Uneven temperature of entire cross section (OJTEMP1)

.

Determined according to EN 1991-1-5 § 6.1.4.1. When assessing the impact, a coating with a thickness of 110 mm is applied on the safe side.

$k_{1.sur}(t = 100mm) = 0.7$

$k_{1.sur}(t = 150mm) = 0.5$

$\rightarrow \; k_{1.sur}(t = 100mm) = 0.70$

$k_{2.sur}(t = 100mm) = 1.0$

$\Delta T_{max} = +15°C \cdot k_{1.sur} = +11°C$: : upper surface warmer

$\Delta T_{min} = -8°C \cdot k_{2.sur} = -8°C$: : lower surface warmer

The occurring temperature change ΔT refers to the linear difference between the temperature at the top and bottom of the bridge deck slab.

Uneven temperature is indicated as a temperature gradient $\frac{\delta T}{\delta Z}$ when defined in FEM-program.

The load effect is added to longitudinal beams with varying height (D: 0.90 m → 1.40), see page A2:24. On safe side δZ = 0.9 m is assumed.

$\frac{\delta T^{max}}{\delta Z} = \frac{+11°C}{0.90m} = +12\frac{°C}{m}$: maximal temperature gradient

$\frac{\delta T^{min}}{\delta Z} = \frac{-8°C}{0.90m} = -9\frac{°C}{m}$: minimal temperature gradient

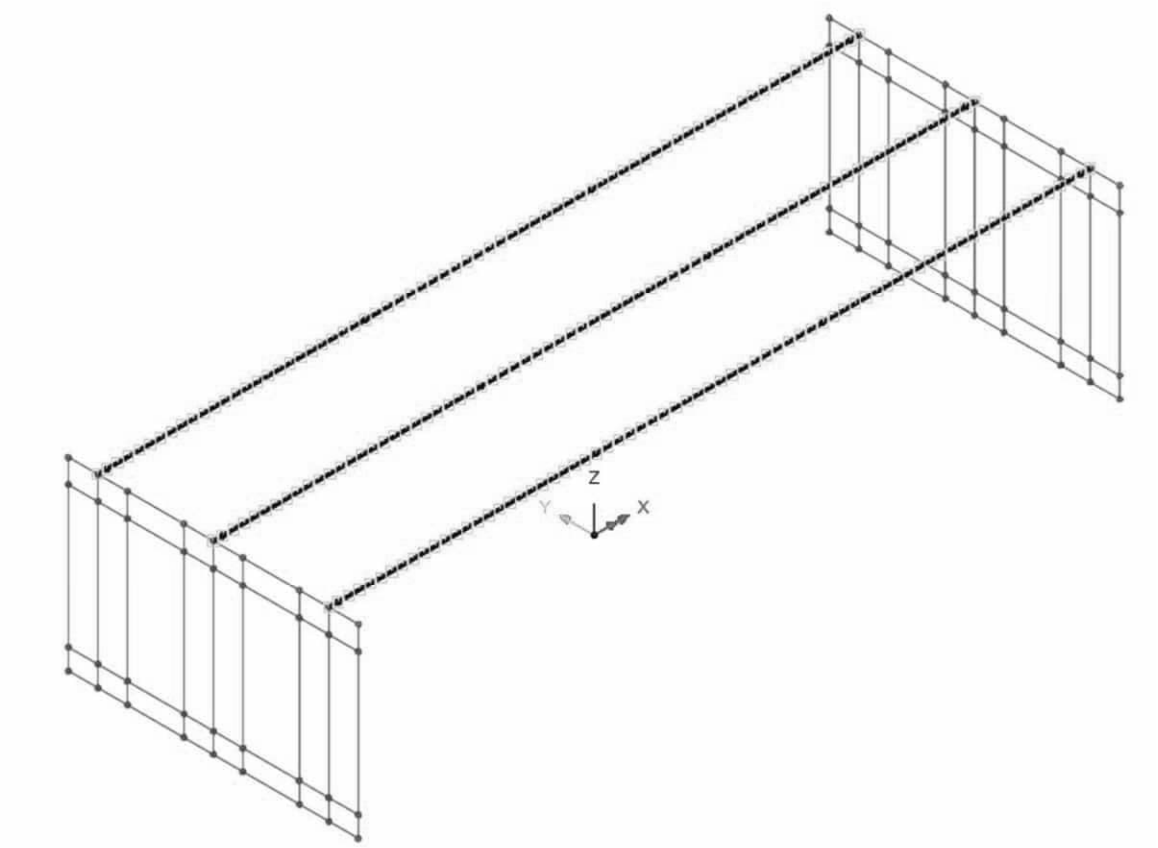
Load case : OJTEMP 1+

Structural loading : Temperature

Definition : Element

Final Z temperature gradient : +12 °C/m

Initial Z temperature gradient : 0°C/m



Overview 3D

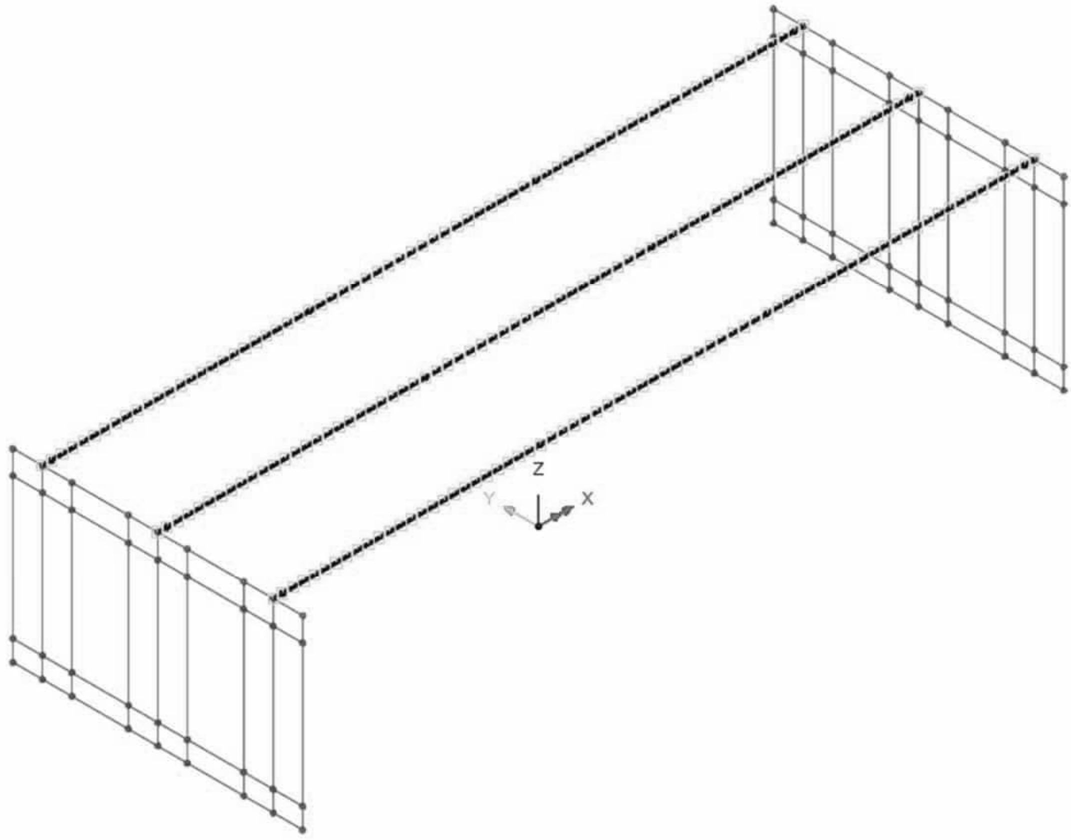
Load case : OJTEMP 1-

Structural loading : Temperature

Definition : Element

Final Z temperature gradient : -9 °C/m

Initial Z temperature gradient : 0°C/m



Overview 3D

Envelope OJTEMP 1 :

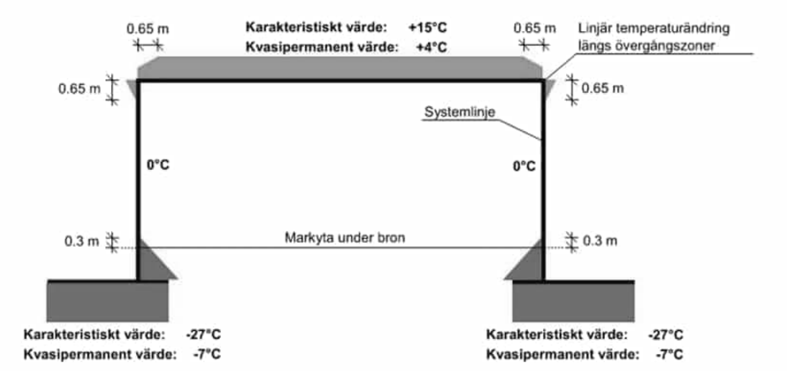
Load case
OJTEMP 1+
OJTEMP 1-

3.12.5 Uneven temperature differences between different construction parts (OJTEMP 2)

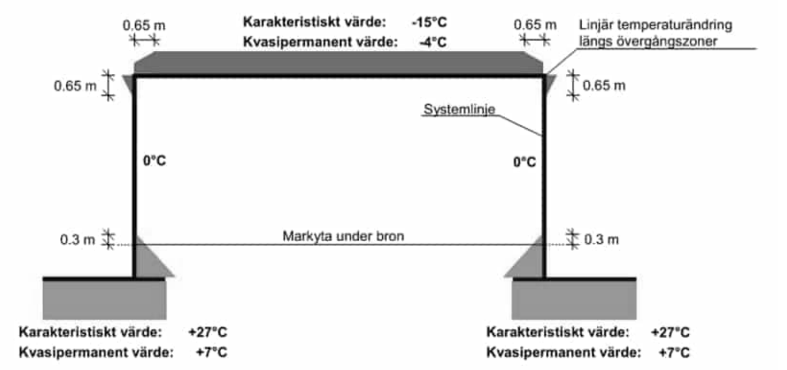
This section states that these effects should be combined with those caused by uniform temperature across the entire bridge (JTEMP).

Recommended values should, according to TRVINFRA-00227 section 2.1.1.2.4, be obtained from TVBK-0373. See extract below.

In TRVINFRA-00227 section 2.1.1.2.4, it is stated that no reduction should be made considering creep, but consideration should be made for cracking.



Principfigur T(+)



Principfigur T(-)

Applied temperature differences between superstructure and abutments:

$\Delta T = \pm 4°C$: SLS-Q

$\Delta T = \pm 15°C$: SLS-K, SLS-F and ULS

Remark
Since temperature load is not considered in the ultimate limit state (ULS).

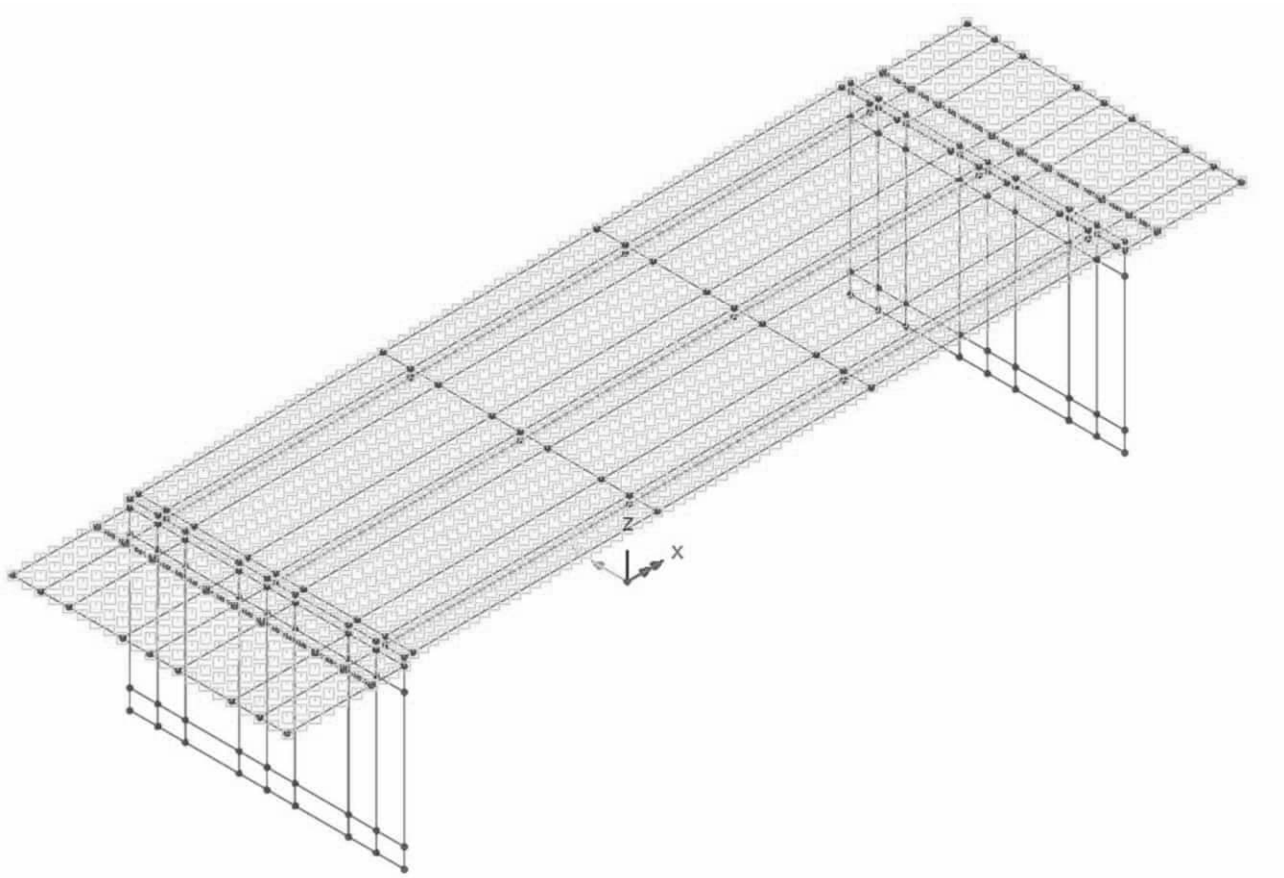
Load : OJTEMP 2

Structural loading : Temperature

Final temperature : +15 C

Initial temperature : ±0 C

Load case : OJTEMP 2+



Overview 3D

Basic load cases :

Load case	Load	Factor
OJTEMP 2-	OJTEMP 2+	-1

Envelope OJTEMP *2:*

Load case
OTJEMP 2+
OJTEMP 2-

3.12.6 Impact of uniform temperature change

Load combination basic JTEMP MAX :

Load case	Factor
JTEMP+	1.00

Load combination basic JTEMP MIN :

Load case	Factor
JTEMP-	1.00

Envelope JTEMP :

Load case
JTEMP MAX
JTEMP MIN

3.12.6.1 Combining load case JTEMP and OJTEMP 1

Load combination is conducted according to SS-EN 1991-1-5, section 6.1.5. For such a combination, $\omega_M = 0.75$ och $\omega_N = 0.35$ shall be applied as shown below.

Alternative 1 ($\omega_M = 0.75$) : $T + \omega_M \cdot \Delta T$

Alternative 2 ($\omega_N = 0.35$) : $\omega_N \cdot T + \Delta T$

Load combination smart TEMP-1 :

Loadcase	Permanent factor	Variable factor
JTEMP	0	0.47 (= 0.77[1.)] x 0.6[2.)] x 1.00)
OJTEMP 1	0	0.45 (= 1.00[1.)] x 0.6[2.)] x 0.75)
DELTA-P	0	1.0

Load combination smart TEMP-2 :

Loadcase	Permanent factor	Variable factor
JTEMP	0	0.16 (= 0.77[1.)] x 0.6[2.)] x 0.35)
OJTEMP 1	0	0.60 (= 1.00[1.)] x 0.6[2.)] x 1.00)
DELTA-P	0	1.0

Note:

[1.)] Impact of creep results in reduced rigidity, see page A3:46.

[2.)] Impact of cracking results in reduced rigidity, see page A3:115.

3.12.6.2 Combining load case JTEMP and OJTEMP 2

Load combination smart TEMP-3 (SLS-F & SLS-K) :

Load case	Permanent factor	Variable factor
JTEMP	0	0.46 (= 0.77[1.)] x 0.6[2.)])
OJTEMP 2	0	0.60 (= 1.00 x 0.6[2.)])
DELTA-P	0	1.0

Note:

[1.)] Impact of creep results in reduced rigidity, see page A3:46.

[2.)] Impact of cracking results in reduced rigidity, see page A3:115.

Load combination smart TEMP-4 (SLS-Q) :

Load case	Permanent factor	Variable factor
JTEMP	0	0.46
OJTEMP 2	0	0.16 (= 0.60 x 4°C/ 15°C)
DELTA-P	0	1.0

3.12.6.3 Summary load combination limit service state (SLS-F & SLS-K)

Envelope TEMP:

Load case
TEMP-1
TEMP-2
TEMP-3

3.12.6.4 Summary load combination limit service state (SLS-Q)

Envelope TEMP-SLS Q:

Load case
TEMP-1
TEMP-2
TEMP-4

3.12.6.5 Summary load combination ultimate limit state (ULS)

Basic load combination TEMP-ULS:

Load case
DELTA P

3.13 PRESTESS

Load applied to Analysis : *Analysis 1*

Analysis of pre tensioned cable is studied at times : t_0 (5 days), t_1(30 days) and t_2 (120 years).

The preliminary location of cables is determined with program PROG B2.001.

The location is imported as a spread sheet into FEM-program as a tension profile. The location is defined with local coordinates associated to nodal lines (LB 1-3).

Initial prestress loss at time t0 is only due to friction. This is determined with FEM-program and program PROG B2.001.

Determination of time loses (η_t) is made in separate program PROG B2.002. Preliminary analysis will use losses seen below. They will be verified later during detailed design.

Time	η_t	Load combination	Load case
t0	0 %	PT-T0	1.00 x PT-T0
t1	6 %	PT-T1	0.94 x PT-T0
t2	16 %	PT-T2	0.84 x PT-T0

3.13.1 General

Pre tensions system VSL 6-15.

Material :

$f_{p0.1k} = 1640 \ MPa$

$f_{pk} = 1860 \ MPa$

$E_{sk} = 195 \ GPa$

$\mu = 0.18$

$k = 0.005 \cdot \frac{1}{m}$

Casting tube :
80 mm / 86 mm

Slip during locking:
6 mm

Permissible curvature :

$R_{\min} = 5.7 m$

Cabel area :

$A_p = 15 \cdot 150 \ mm^2 = 2250 \ mm^2$

Anchor plate :
290 mm x 290 mm (same for both passiv and active anchorage)

Ultimate load :

$F_u = 2250 \ mm^2 \cdot 1860 \ MPa = 4185 \ kN$

Permissible stress before locking :
See SS-EN 1992-1-1 section 5.10.2.1

$\sigma_{p,\max}^{före} = \min(0.8 f_{pk} ; 0.9 f_{p0.1k}) = \min(1488 \ MPa ; 1476 \ MPa) = 1476 \ MPa$

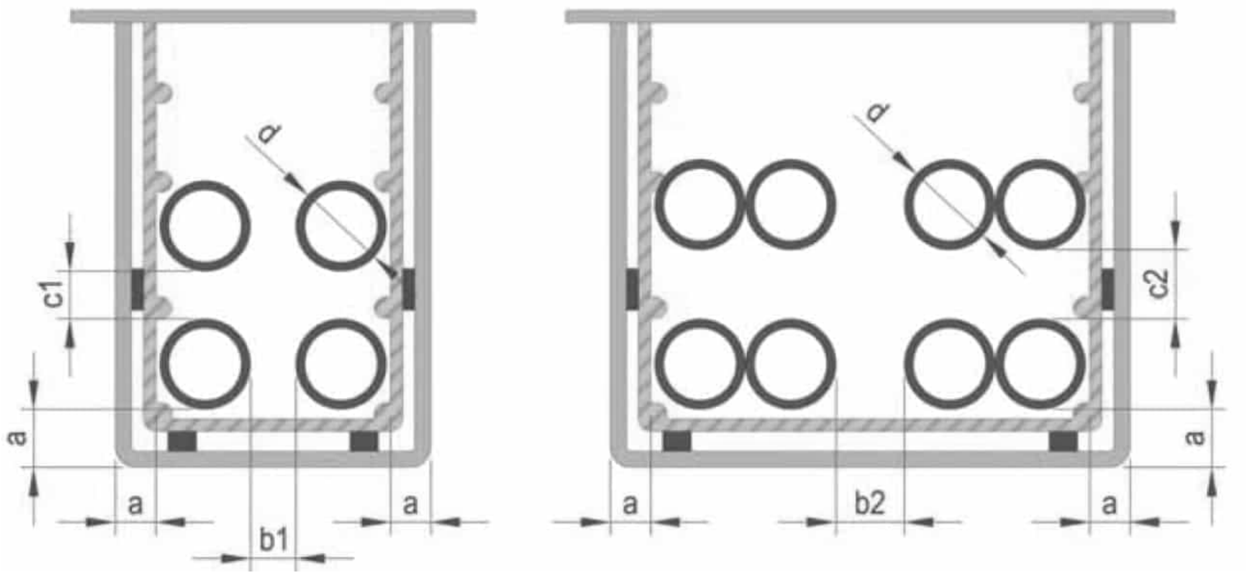
Permissible stress after locking :
See SS-EN 1992-1-1 section 5.10.3

$\sigma_{p,\max}^{efter} = \min(0.75 f_{pk} ; 0.85 f_{p0.1k}) = \min(1395 \ MPa ; 1394 \ MPa) = 1394 \ MPa$

3.13.2 Execution

Associated to pre tension system VSL 6-15.

Recommended measurements :



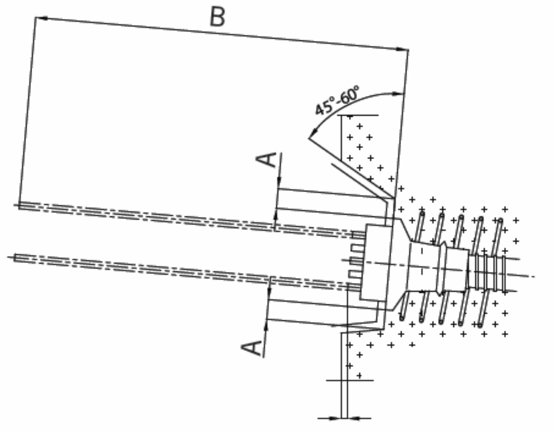
d = 90 mm

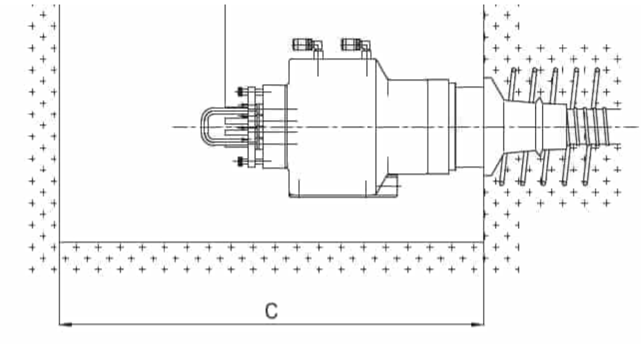
a > 50 mm

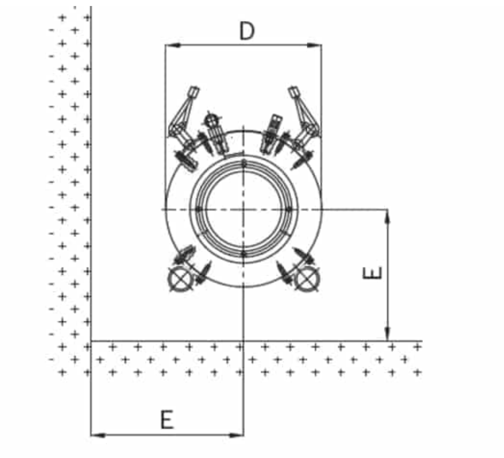
b1, c1 > 0.7d = 63 mm but 100 chosen !

b2, c2 > 1.0d = 90 mm but 100 mm chosen !

Demand for space during tensioning :





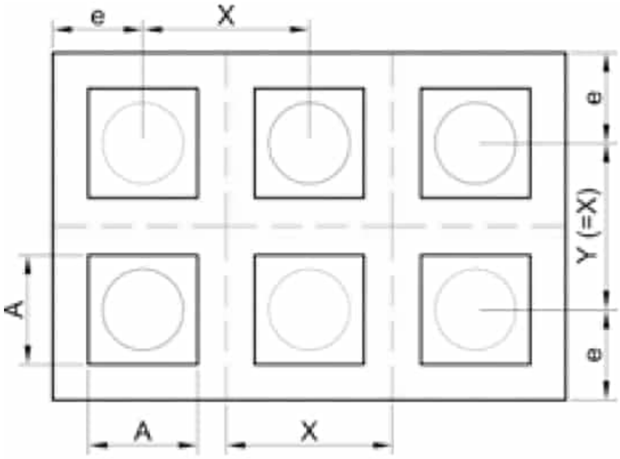


A = 70 mm

B = 1200 mm

C = 1700 mm

Measurements of cables VSL 12-15 :



A = 290 mm

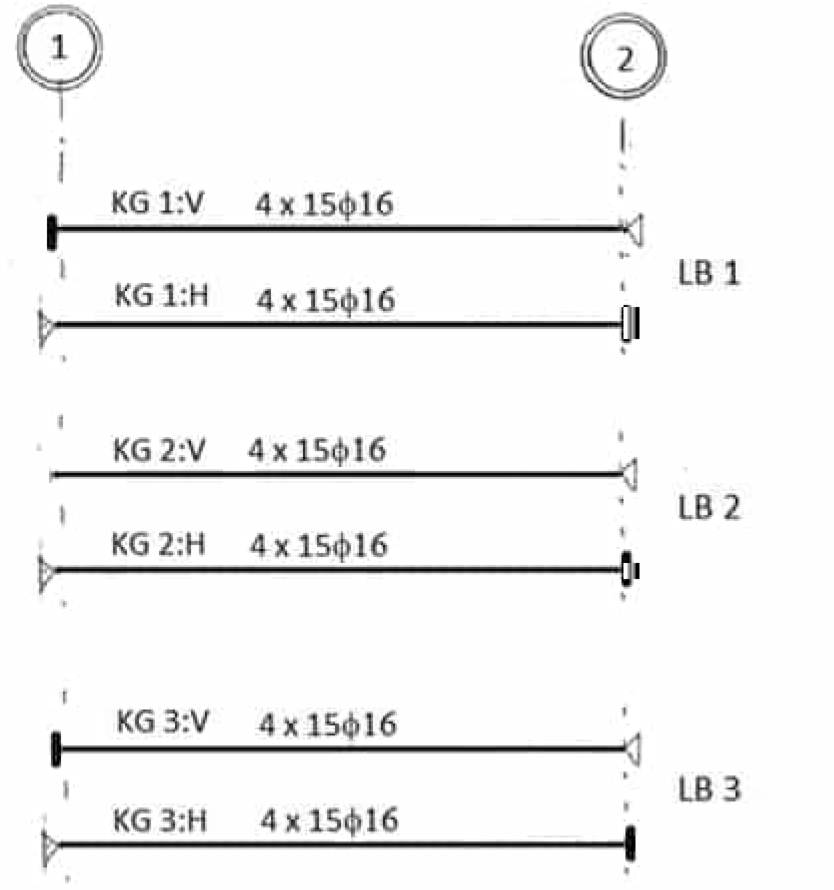
e ≥ 175 mm + TB

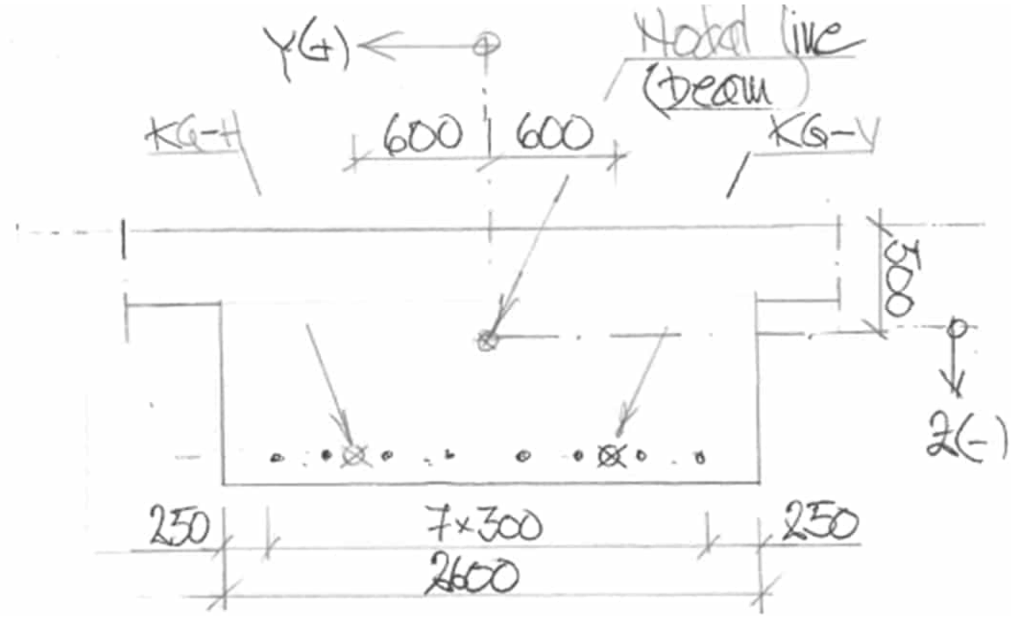
X ≥ 400 mm

3.13.3 Preliminary cable location

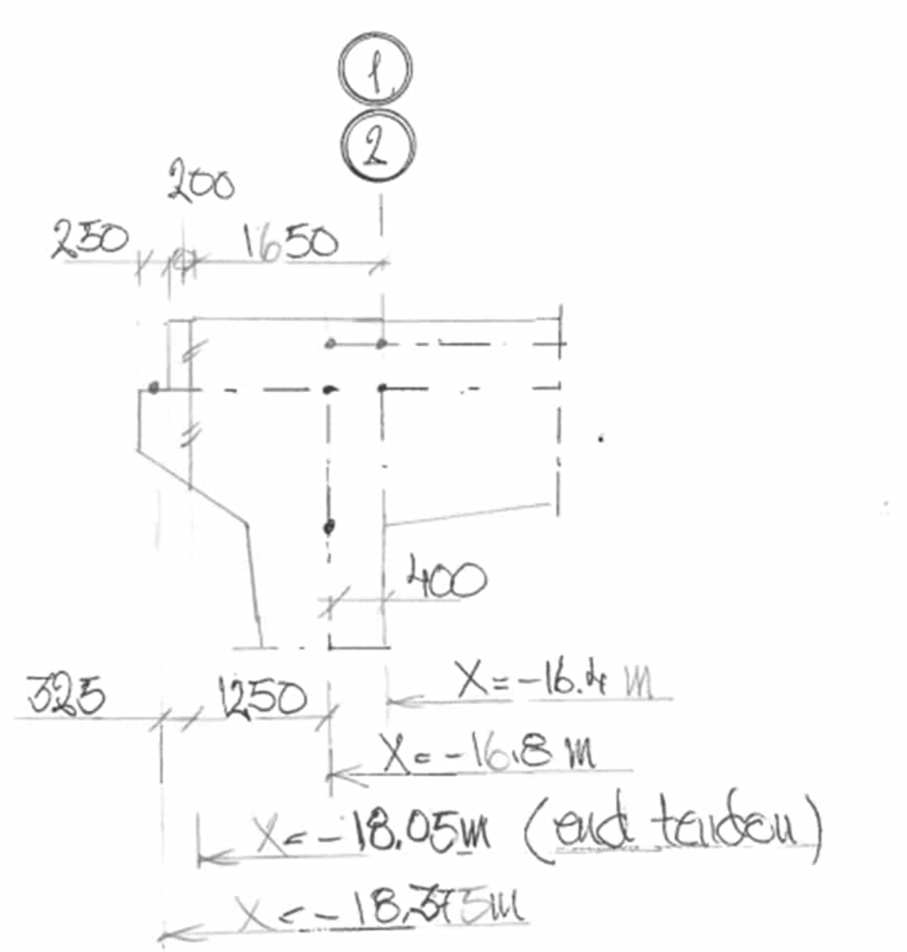
In the static model cables are simplified (= 4 cables are modelled as one fictive cable as seen below).

Profiles can be defined using "global coordinates" or as "local coordinates mapped to lines". The later of this method is used. The nodal lines associated to LB 1, LB 2 and LB 3 are used.



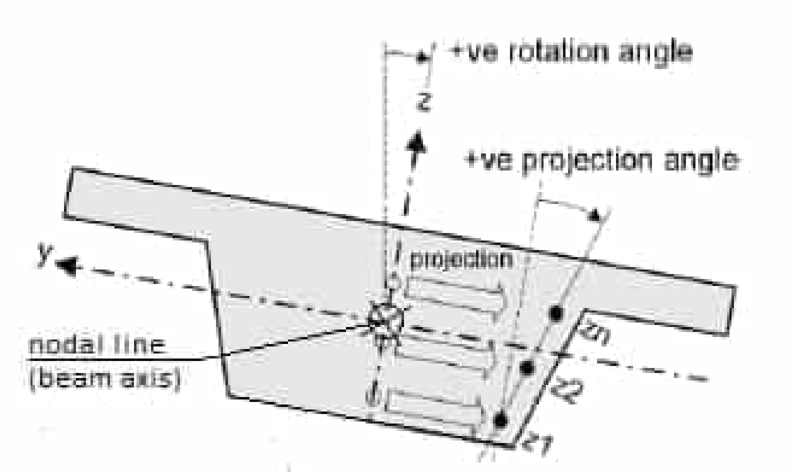


Cross section



Detail end of tendon

Principle sketch:



Summary input - pretensioned cables:

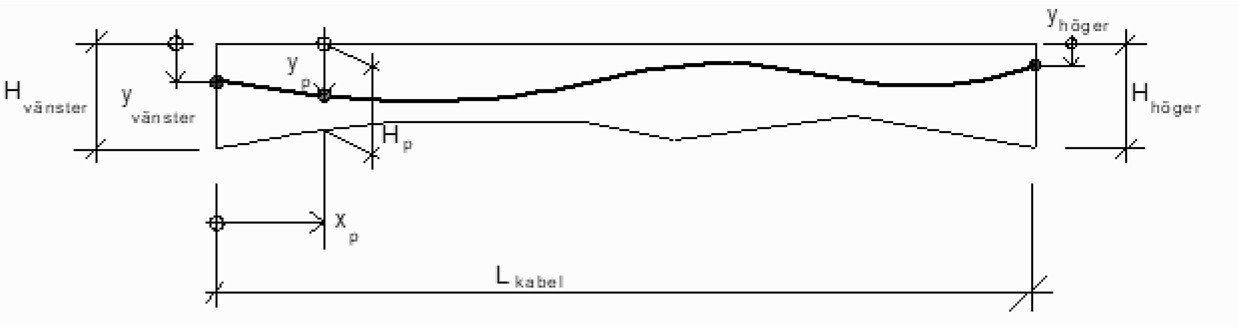
Cable	Area	Slip	Left side	Right side	Max. prestress before	Min. prestress after	Pretension force	Location y(+)
KG-V	2250	6	Active	Active	1476	1394	3150*	+0.60
KG-H	2250	6	Active	Active	1476	1394	3150*	-0.60
-	mm^2	mm	-	-	MPa	MPa	kN	m

* = chosen prestress 1400 MPa

Beam	Location	Cables	Fictive load
LB 1	Left side	4	4 x KG-V
-“-	Right side	4	4 x KG-H
LB 2	Left side	4	4 x KG-V
-“-	Right side	4	4 x KG-H
LB 3	Left side	4	4 x KG-V
-“-	Right side	4	4 x KG-H
-	-	-	mm^2

Object: Tendon profile (1 cable VSL 6-15)

PRINCIPLE SKETCH



INPUT

Total cable length: $L_{kabel} := 1.65 \cdot m + 32.8 \cdot m + 1.65\ m = 36.1\ m$

Number of definition points: $N := 11 \cdot st$

Friction coefficients: $\mu := 0.18$ $k := 0.005 \cdot \frac{rad}{m}$

Resistance cable: $f_{p0.1k} := 1640 \cdot MPa$

$f_{pk} := 1860 \cdot MPa$

E-modulus cable: $E_s := 195 \cdot GPa$

Tendon area: $A_s := 2250 \cdot mm^2$

Maximum permissible tensile force before locking according to SS-EN 1992-1-1 section 5.10.2.1:

$$min\left(0.8 \cdot f_{pk}, 0.9 \cdot f_{p0.1k}\right) \cdot A_s = 3321\ kN$$

Selected prestressing force:

$$V_{ö} := 3320 \cdot kN$$

Maximum permissible tensile force after locking according to SS-EN 1992-1-1 section 5.10.3:

$$min\left(0.75 \cdot f_{pk}, 0.85 \cdot f_{p0.1k}\right) \cdot A_s = 3137\ kN$$

Type of anchorage ("Passiv" or "Aktiv") / chosen maximum tendon force after locking:

Section	Type	V_{max} [kN]
Left	Aktiv	3137
Right	Passiv	3137

Defined points along cable:

Snitt	x_p (m)	y_p (mm)	H_p (mm)
1	0	300	1700
2	1,00	300	1700
3	1,65	320	1700
4	7,05	500	1338
5	13,05	850	1200
6	18,05	900	1200
7	23,05	850	1200
8	29,05	500	1338
9	34,45	320	1700
10	35,10	300	1700
11	36,10	300	1700

LUSAS ⇒

x(+)	z(+)
0	0,20
1,00	0,20
1,65	0,18
7,05	0,00
13,05	-0,35
18,05	-0,40
23,05	-0,35
29,05	0,00
34,45	0,18
35,10	0,20
36,10	0,20
m	m

CALCULATION

Create mathematical functions for a beam and a cable

$C := \text{pspline}\left(x_p, y_p\right)$: determination of coefficients for parabolic spline functions

$y(x) := \text{interp}\left(C, x_p, y_p, x\right)$: cable routing (= spline functions)

$y'(x) := \frac{d}{dx} y(x)$: slope of cable routing

$y''(x) := \frac{d^2}{dx^2} y(x)$: curvature change of cable routing

$R_{min} := \frac{1}{\max\left(y''(x)\right)}$: lowest curvature radius of cable routing

Friction loss function measured from the "left" side

$\alpha_v := \mathbf{if}\left(i > 1, \sum_{j=2}^{i} \left|y'(x_j) - y'(x_{j-1})\right|, 0\right)$: accumulated change in angle

$\beta_v := \mu \cdot \left(\alpha_v + k \cdot x\right)$: friction loss exponent

$\eta_{vf} := e^{-\beta_v}$: friction loss before locking

$\eta_{ve} := e^{\beta_v}$: friction loss after locking

Location of maximal cable force on "left" side after locking of cable

```
X_mv = | x_skär ← 0m  if  Typ = "Passiv"
       | if  Typ = "Aktiv"
       |    | x_start ← 2m
       |    | x_skär ← root(V_max − V_ö·linterp(X, η_vf, x_start), x_start)
```

Friction loss function measured from the "right" side

$$\alpha_h := \text{if}\left(i > 1, \sum_{j=i+1}^{n} \left|y'(x_j) - y'(x_{j-1})\right|, 0\right)$$: accumulated change in angle

$$\beta_h := \mu \cdot \left(\alpha_h + k \cdot \left(L_{kabel} - x\right)\right)$$: friction loss exponent

$$\eta_{hf} := e^{-\beta_h}$$: friction loss before locking

$$\eta_{he} := e^{\beta_h}$$: friction loss after locking

Location of maximal cable force on "right" side after locking of cable

$$X_{mh} = \begin{cases} x_{skär} \leftarrow L_{kabel} \text{ if } Typ = \text{"Passiv"} \\ \text{if } Typ = \text{"Aktiv"} \\ \quad x_{start} \leftarrow L_{kabel} - 2m \\ \quad x_{skär} \leftarrow \text{root}\left(V_{max} - V_{ö} \cdot \text{linterp}\left(X, \eta_{hf}, x_{start}\right), x_{start}\right) \end{cases}$$

Location where curve of cable force "right" side intersects curve of cable force "left"

$$X_m = \begin{cases} x_{skär} \leftarrow L_{kabel} \text{ if } Typ = \text{"Aktiv"} \wedge Typ = \text{"Passiv"} \\ x_{skär} \leftarrow 0m \text{ if } Typ = \text{"Passiv"} \wedge Typ = \text{"Aktiv"} \\ \text{if } Typ = \text{"Aktiv"} \wedge Typ = \text{"Aktiv"} \\ \quad x_{start} \leftarrow 0.5 \cdot L_{kabel} \\ \quad x_{skär} \leftarrow \text{root}\left(V_{ö} \cdot \text{linterp}\left(X, \eta_{vf}, x_{start}\right) - V_{ö} \cdot \text{linterp}\left(X, \eta_{hf}, x_{start}\right), x_{start}\right) \end{cases}$$

Determine cable force at each end of cable after locking

Cable force at "left" side :

$$P_{ve} = \begin{cases} V_{max} \cdot \text{linterp}\left(X, \eta_{hf}, 0m\right) \text{ if } Typ = \text{"Passiv"} \\ \dfrac{V_{max}}{\text{linterp}\left(X, \eta_{ve}, X_{mv}\right)} \text{ if } Typ = \text{"Aktiv"} \end{cases}$$

Cable force at "right" side :

$$P_{he} = \begin{cases} V_{max} \cdot \text{linterp}\left(X, \eta_{vf}, L_{kabel}\right) \text{ if } Typ = \text{"Passiv"} \\ \dfrac{V_{max}}{\text{linterp}\left(X, \eta_{he}, X_{mh}\right)} \text{ if } Typ = \text{"Aktiv"} \end{cases}$$

Determine post slip / "lock sliding" at each end of cable

Left side :

$$\Delta L_v := \textbf{if}\left(Typ = \text{“Aktiv”}, \frac{1}{A_s \cdot E_s} \cdot \int_0^{X_{mv}} \left(V_ö \cdot \text{linterp}\left(X, \eta_{vf}, x\right) - P_{ve} \cdot \text{linterp}\left(X, \eta_{ve}, x\right)\right) \text{d}x, 0\right)$$

Right side :

$$\Delta L_h := \textbf{if}\left(Typ = \text{“Aktiv”}, \frac{1}{A_s \cdot E_s} \cdot \int_{X_{mh}}^{L_{kabel}} \left(V_ö \cdot \text{linterp}\left(X, \eta_{hf}, x\right) - P_{ve} \cdot \text{linterp}\left(X, \eta_{he}, x\right)\right) \text{d}x, 0\right)$$

Determine cable elongation before locking of cable

Left side:

$$L_v := \frac{1}{A_s \cdot E_s} \cdot \int_0^{X_m} \left(V_ö \cdot \text{linterp}\left(X, \eta_{vf}, x\right)\right) \text{d}x$$

Right side :

$$L_h := \frac{1}{A_s \cdot E_s} \cdot \int_{X_m}^{L_{kabel}} \left(V_ö \cdot \text{linterp}\left(X, \eta_{hf}, x\right)\right) \text{d}x$$

Function - determine cable force at arbitrary location along cable before locking

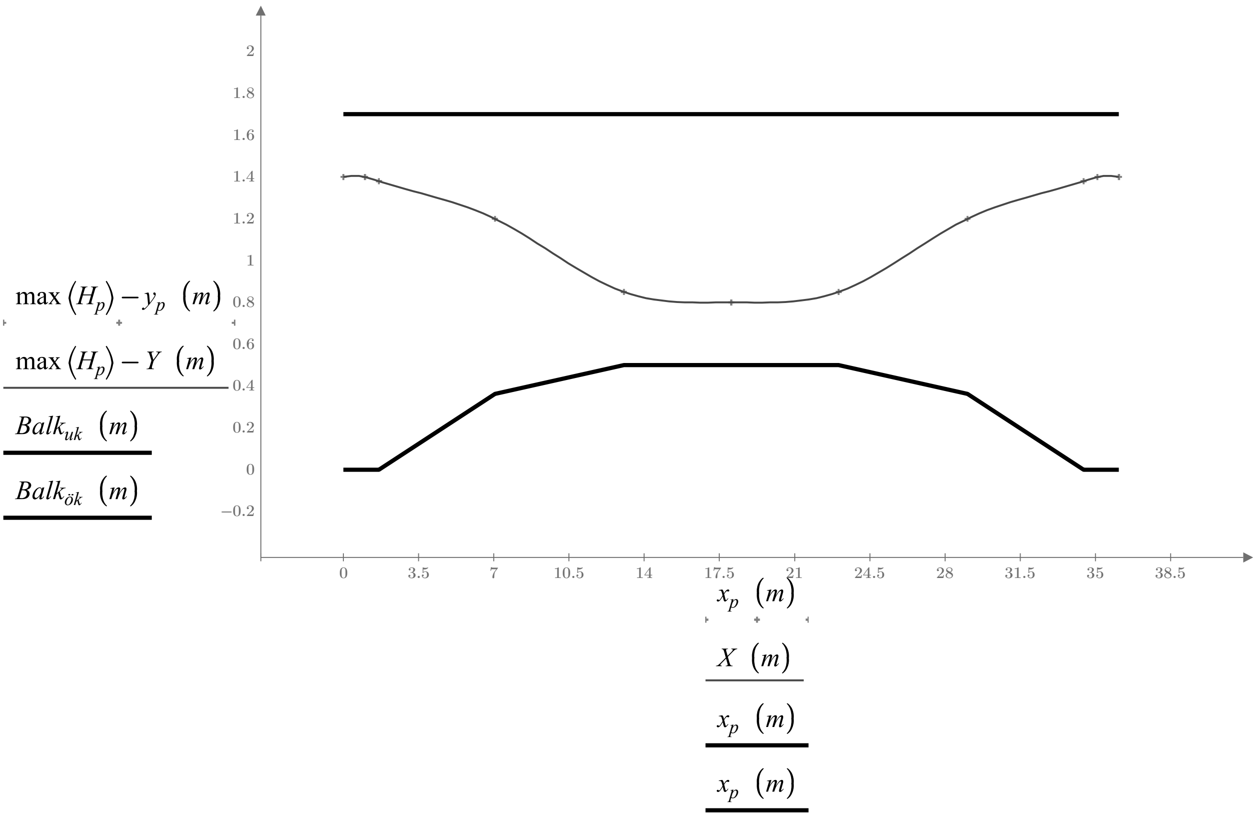
$$P_{före} = \begin{cases} V_ö \cdot \text{linterp}(X, \eta_{vf}, x) & \text{if } x \le X_m \\ V_ö \cdot \text{linterp}(X, \eta_{hf}, x) & \text{if } x > X_m \end{cases}$$

Function - determine cable force at arbitrary location along cable after locking

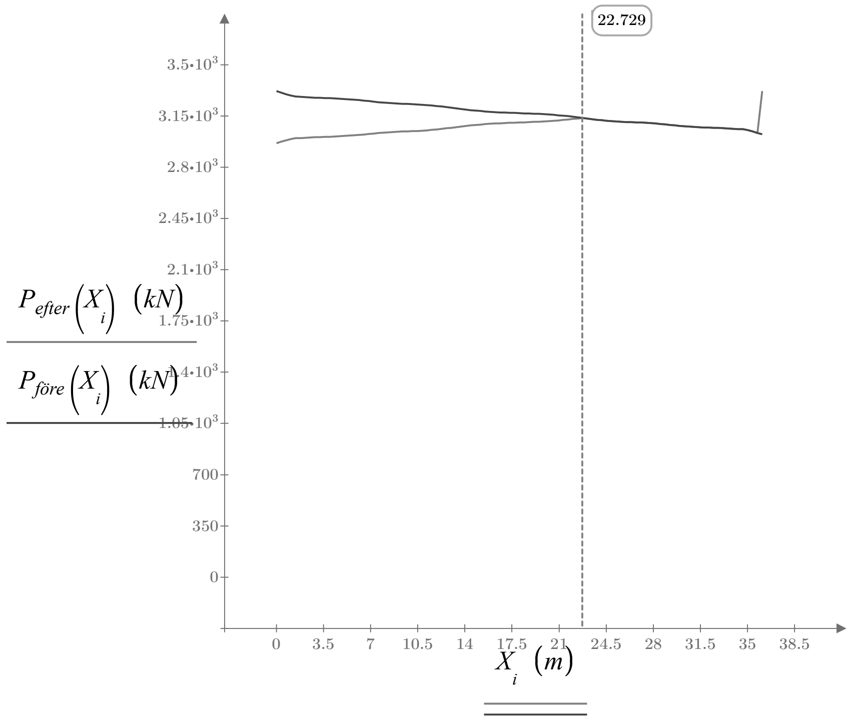
$$P_{efter} := \begin{cases} P_{ve} \cdot \text{linterp}\left(X, \eta_{ve}, x\right) & \text{if } x \le X_{mv} \\ V_ö \cdot \text{linterp}\left(X, \eta_{vf}, x\right) & \text{if } X_{mv} < x < X_m \\ V_ö \cdot \text{linterp}\left(X, \eta_{hf}, x\right) & \text{if } X_m \le x \le X_{mh} \\ P_{he} \cdot \text{linterp}\left(X, \eta_{he}, x\right) & \text{if } x > X_{mh} \end{cases}$$

RESULTS

Beam and Cable routing — graphic presentation



Graphical plotting of cable forces



Minimum curvature radius

$R_{min} = 22.7\ m$

Cable elongation before locking

$L_v = 261\ mm$: left side

$L_h = 0\ mm$: right side

Post slip / "lock sliding" at each end of cable

$\Delta L_v = 8\ mm$: left side

$\Delta L_h = 0$: right side

Cable force at each end after locking

$P_{ve} = 2964\ kN$: left side

$P_{he} = 2858\ kN$: right side

Location of maximum cable force after locking

$X_{mv} = 22.729\ m$: left side

$X_{mh} = 36.1\ m$: right side

Location of minimum cable force after locking

$X_m = 36.1\ m$

3.13.4 Load definition

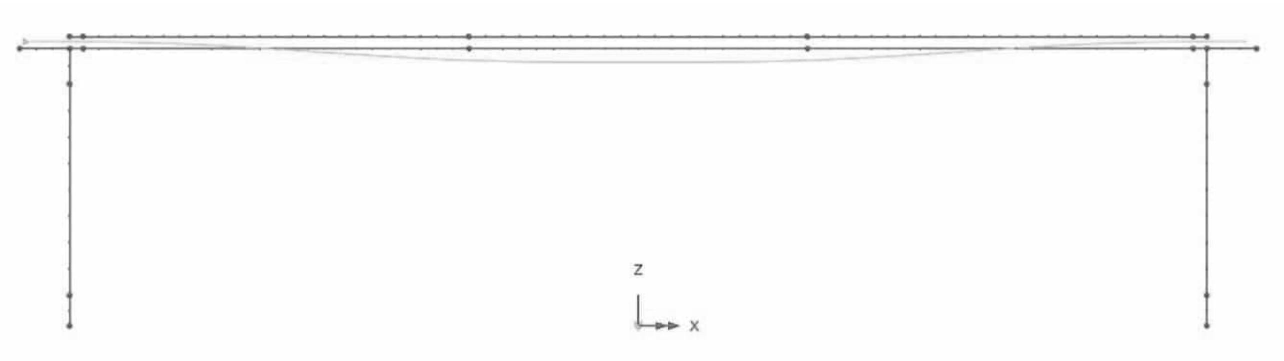
Spread sheet input (see page A3:146):

Offset from end 1 : 0.325 m

x(+) : $x_p(+)$

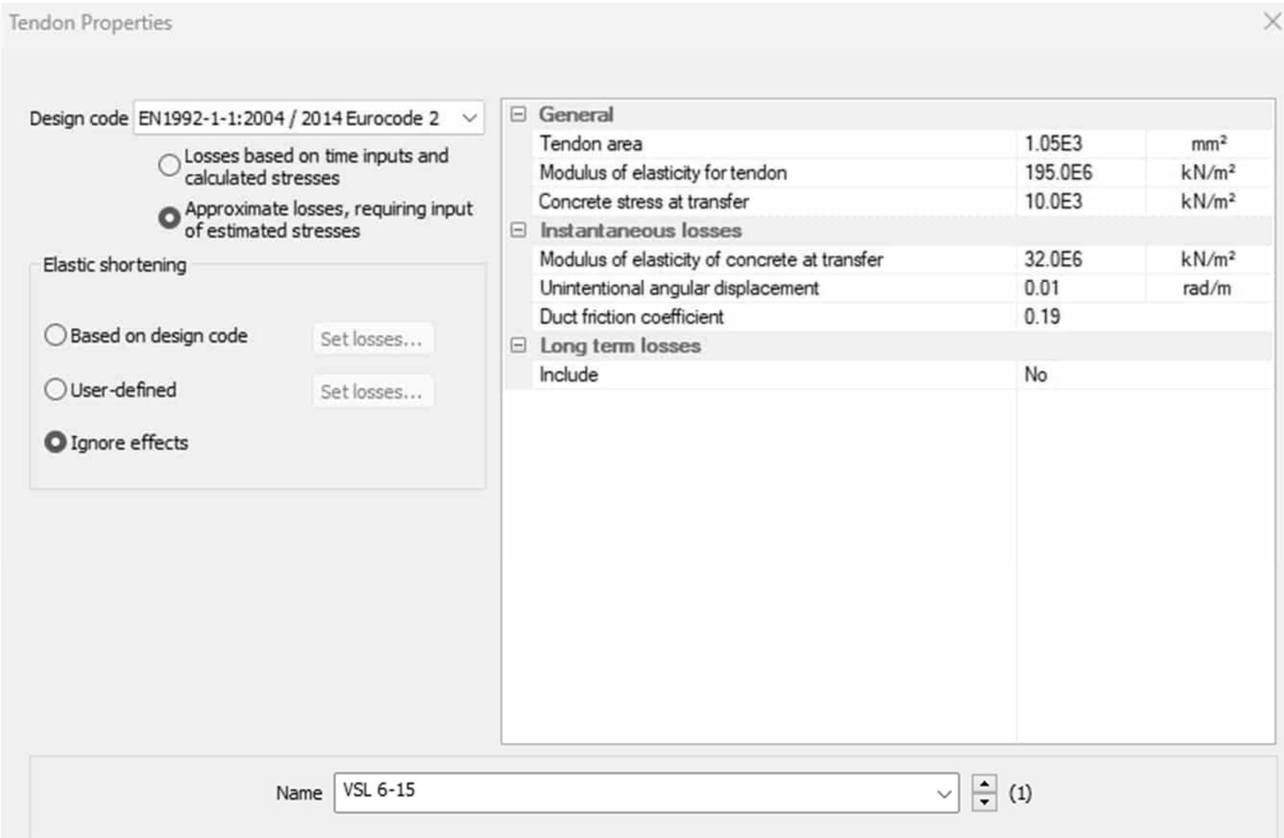
z(+): 0.50 m - $y_p(+)$

x(+)	z(+)
0,000	0,200
1,000	0,200
1,650	0,180
7,050	0
13,050	-0,350
18,050	-0,400
23,050	-0,350
29,050	0
34,450	0,180
35,100	0,200
36,100	0,200
m	m

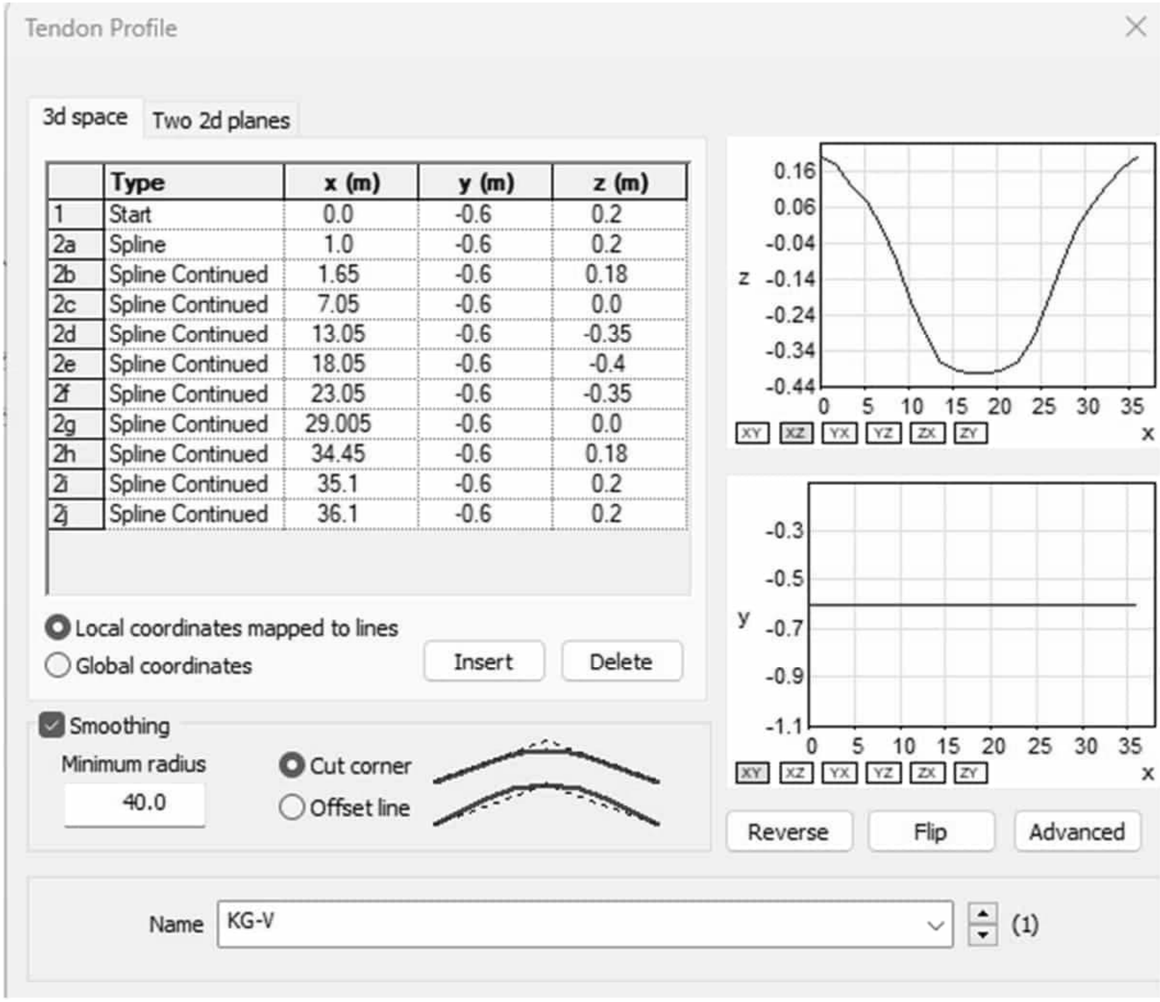


ELEVATION

Tendon properties:



Tendon profiles:

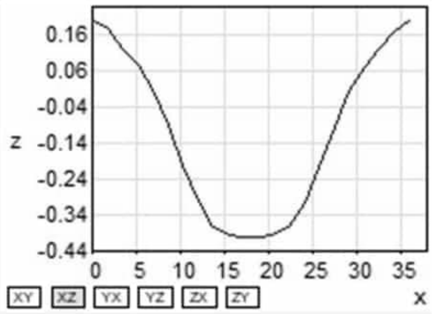


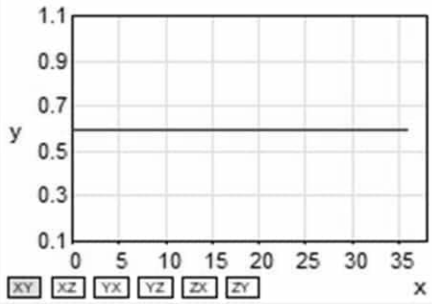
	Type	x (m)	y (m)	z (m)
1	Start	0.0	-0.6	0.2
2a	Spline	1.0	-0.6	0.2
2b	Spline Continued	1.65	-0.6	0.18
2c	Spline Continued	7.05	-0.6	0.0
2d	Spline Continued	13.05	-0.6	-0.35
2e	Spline Continued	18.05	-0.6	-0.4
2f	Spline Continued	23.05	-0.6	-0.35
2g	Spline Continued	29.005	-0.6	0.0
2h	Spline Continued	34.45	-0.6	0.18
2i	Spline Continued	35.1	-0.6	0.2
2j	Spline Continued	36.1	-0.6	0.2

Tendon Profile

3d space | Two 2d planes

	Type	x (m)	y (m)	z (m)
1	Start	0.0	0.6	0.2
2a	Spline	1.0	0.6	0.2
2b	Spline Continued	1.65	0.6	0.18
2c	Spline Continued	7.05	0.6	0.0
2d	Spline Continued	13.05	0.6	-0.35
2e	Spline Continued	18.05	0.6	-0.4
2f	Spline Continued	23.05	0.6	-0.35
2g	Spline Continued	29.05	0.6	0.0
2h	Spline Continued	34.45	0.6	0.18
2i	Spline Continued	35.1	0.6	0.2
2j	Spline Continued	36.1	0.6	0.2





(•) Local coordinates mapped to lines
( ) Global coordinates

Insert | Delete

[x] Smoothing

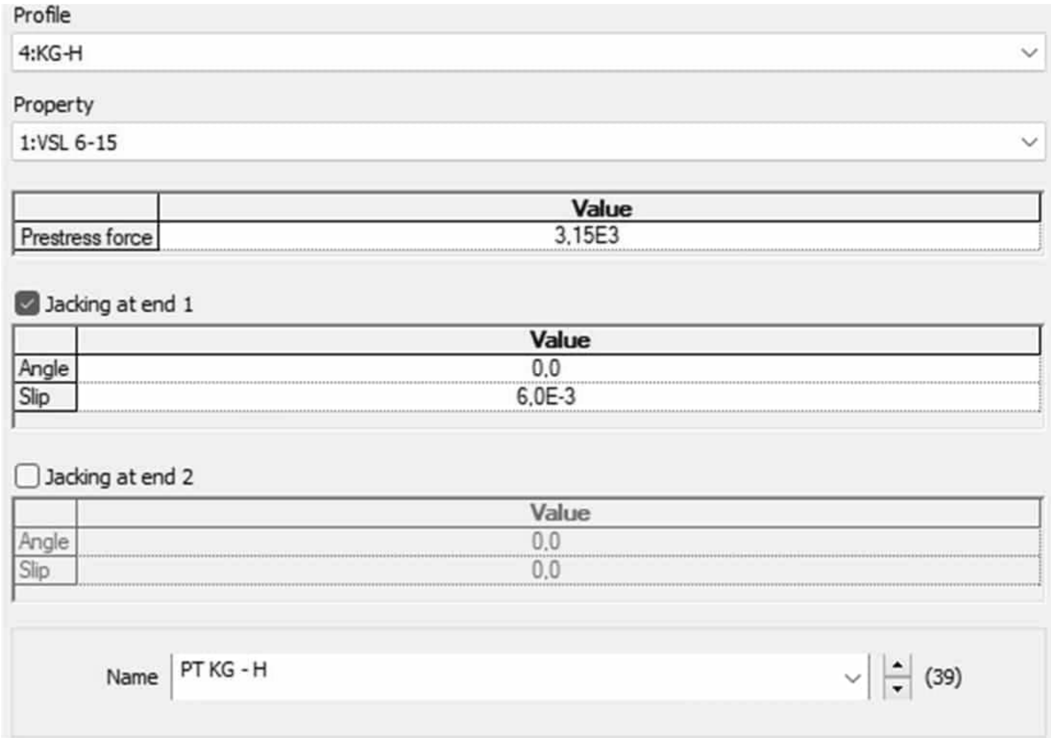
Minimum radius 40.0

(•) Cut corner
( ) Offset line

Reverse | Flip | Advanced

Name KG-H (4)

Tendon loads:



Profile

4:KG-H

Property

1:VSL 6-15

	Value
Prestress force	3,15E3

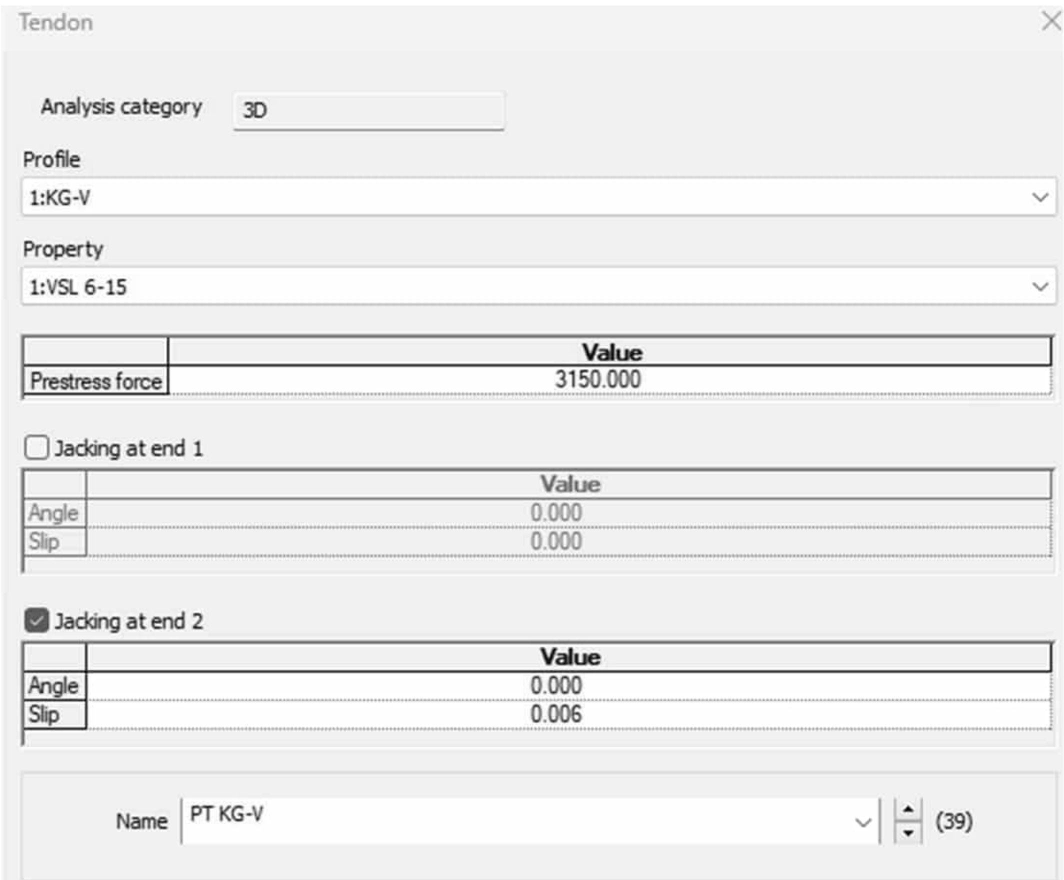
☑ Jacking at end 1

	Value
Angle	0,0
Slip	6,0E-3

☐ Jacking at end 2

	Value
Angle	0,0
Slip	0,0

Name PT KG - H (39)



Tendon

Analysis category 3D

Profile

1:KG-V

Property

1:VSL 6-15

	Value
Prestress force	3150.000

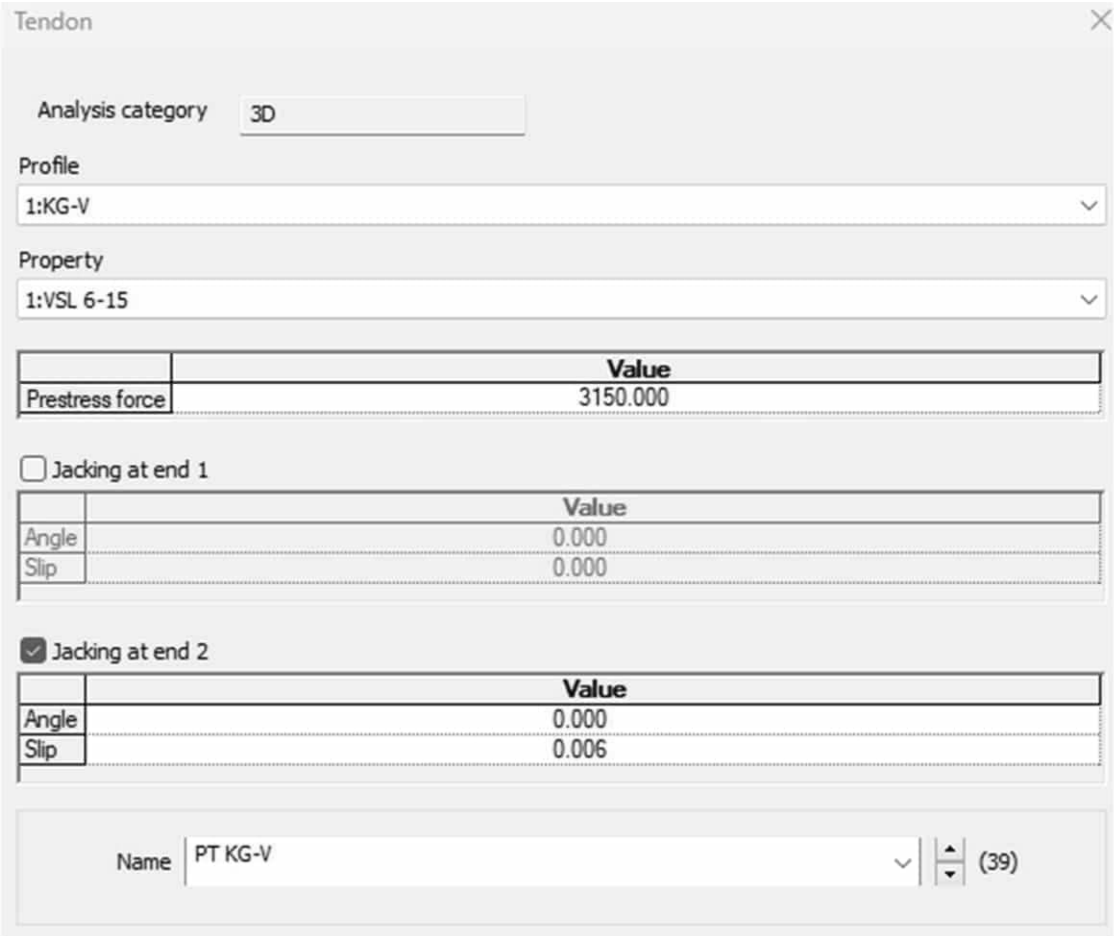
☐ Jacking at end 1

	Value
Angle	0.000
Slip	0.000

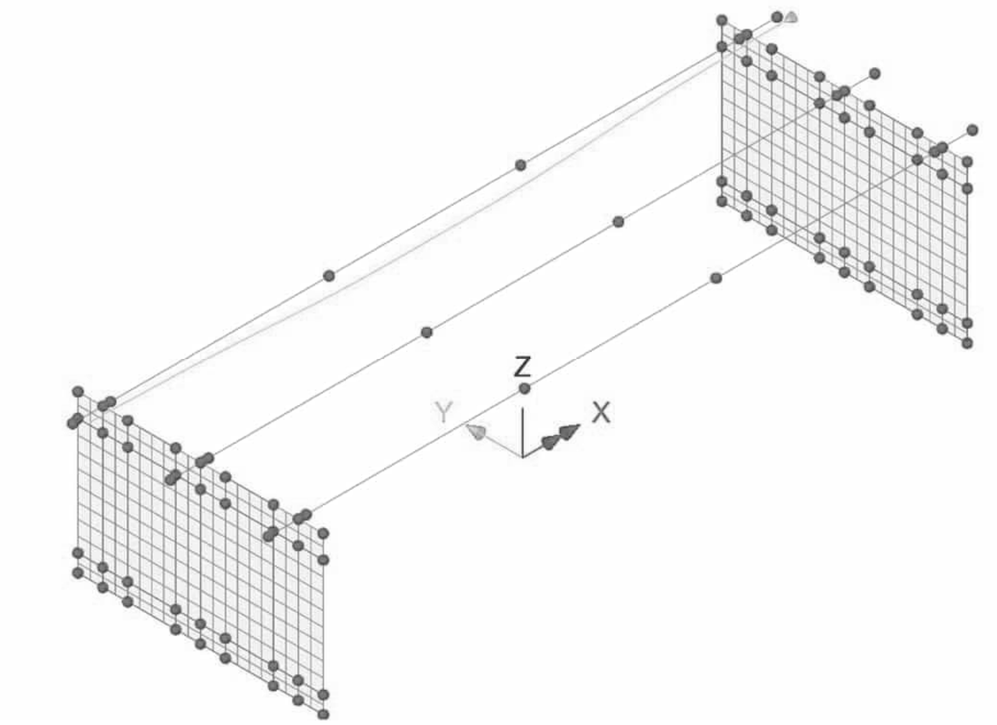
☑ Jacking at end 2

	Value
Angle	0.000
Slip	0.006

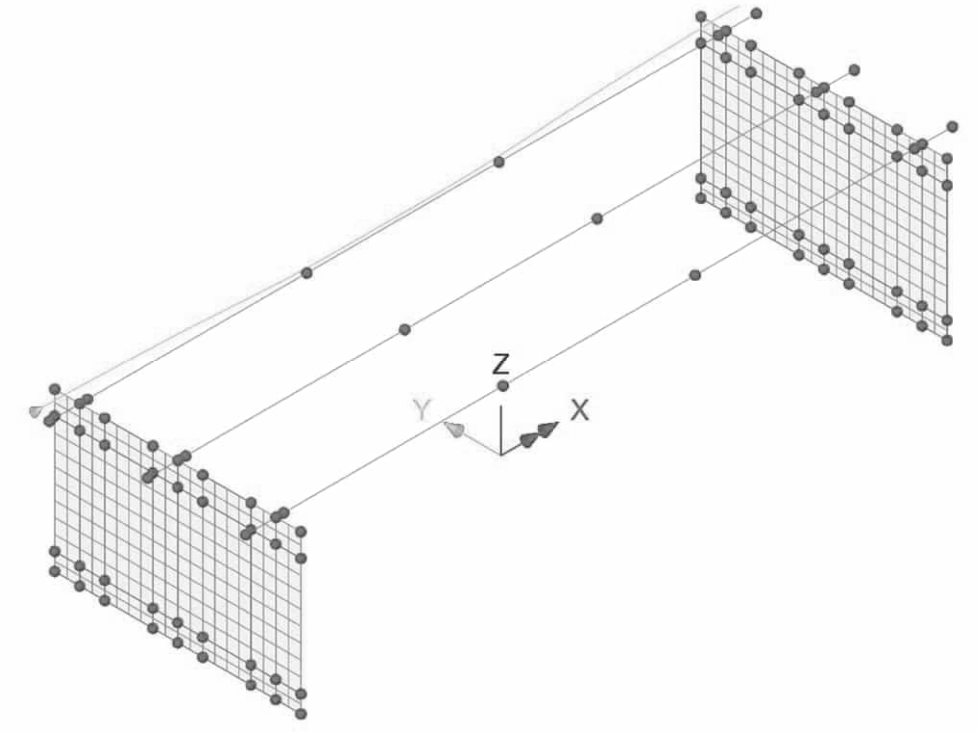
Name PT KG-V (39)



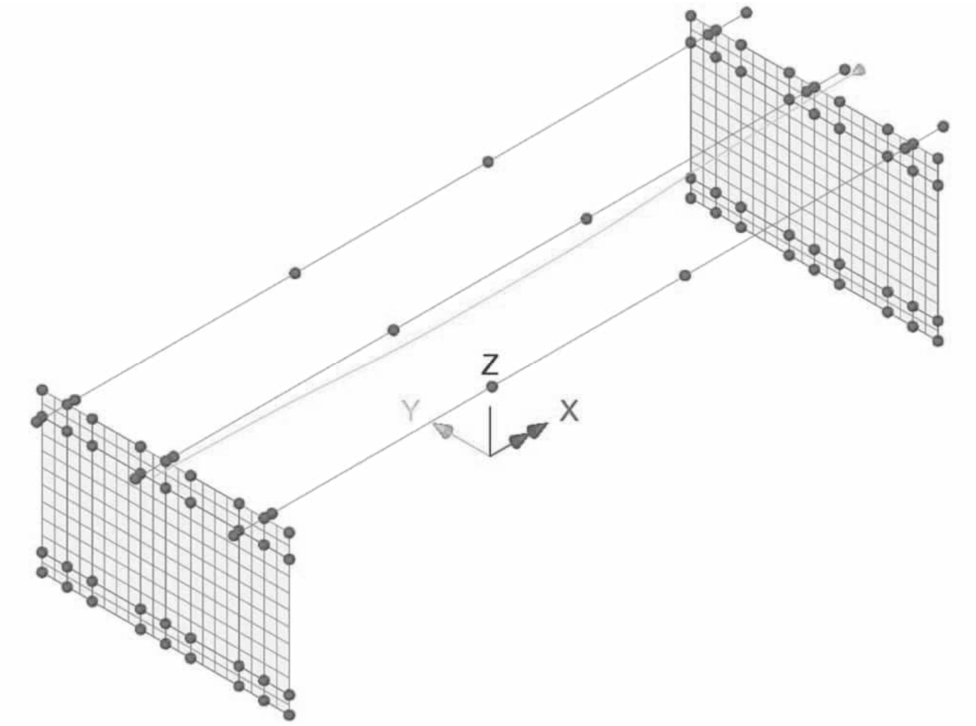
Tendon
Analysis category
3D
Profile
1:KG-V
Property
1:VSL 6-15
Value
Prestress force
3150.000
Jacking at end 1
Value
Angle
0.000
Slip
0.000
Jacking at end 2
Value
Angle
0.000
Slip
0.006
Name
PT KG-V
(39)



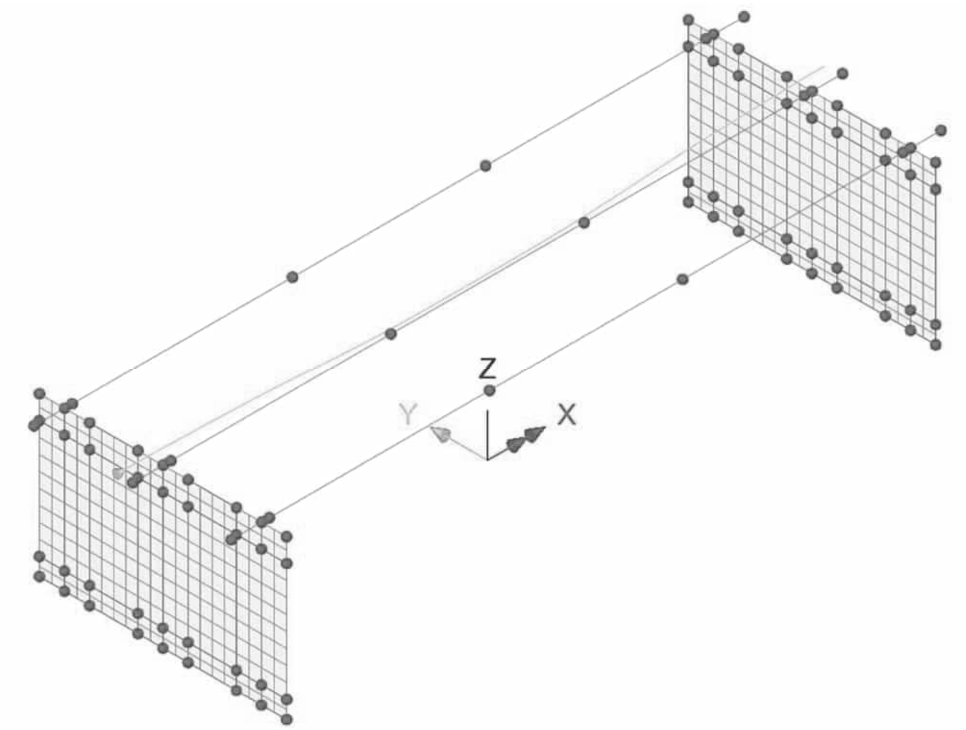
Load case: PT KG V1
(Tendons applied to beam L1)



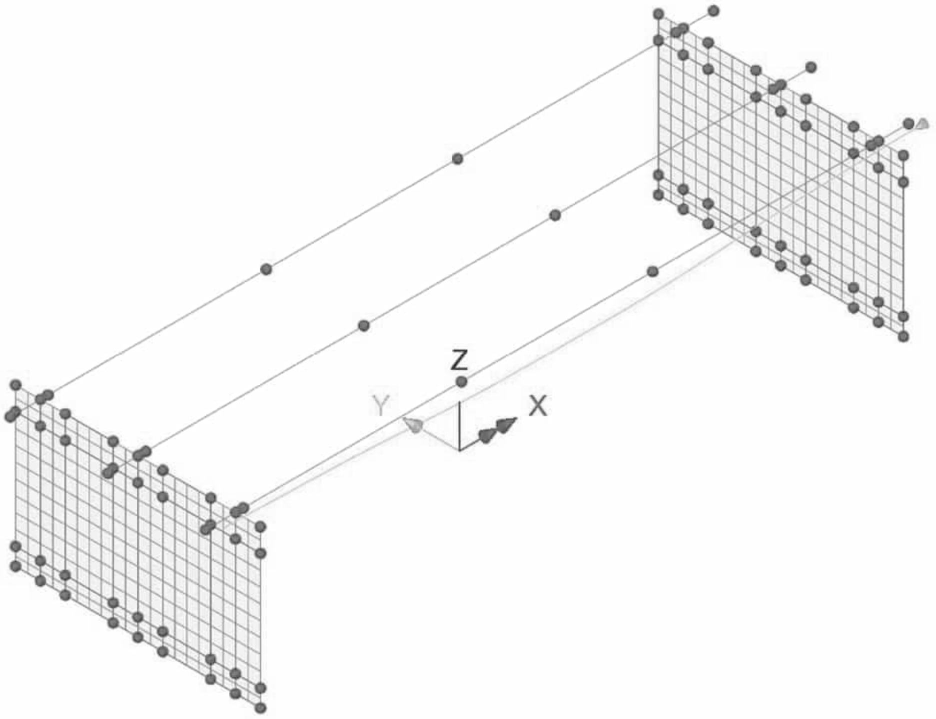
Load case: PT KG H1
(Tendons applied to beam L1)



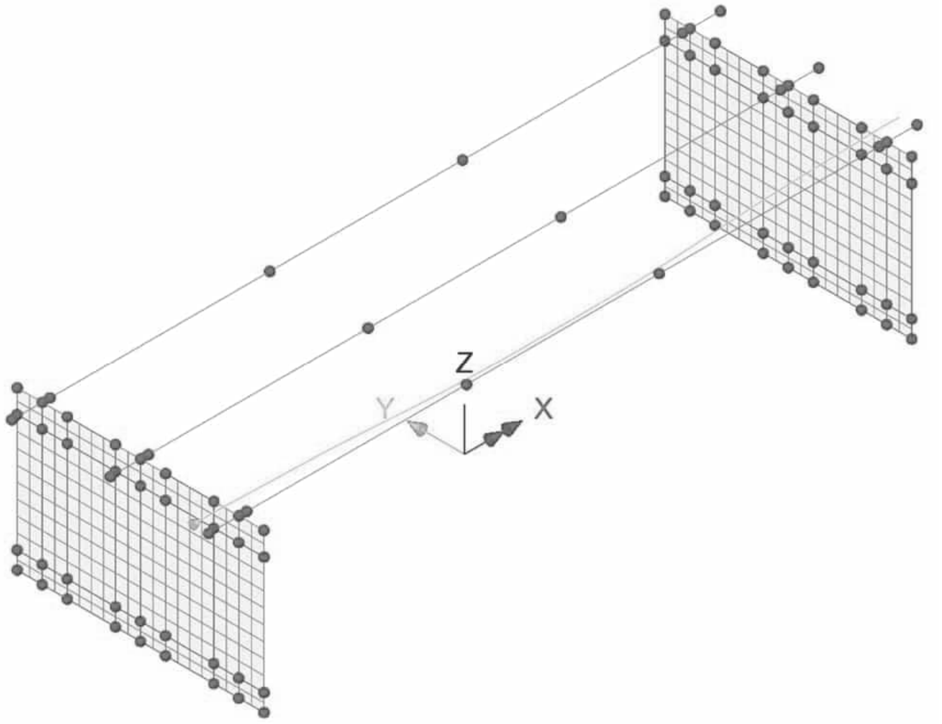
Load case: PT KG V2
(Tendons applied to beam L2)



Load case: PT KG H2
(Tendons applied to beam L2)



Load case: PT KG V3
(Tendons applied to beam L3)



Load case: PT KG H3
(Tendons applied to beam L3)

3.13.4 Load combination

Load combination basis PT-t0:

Load case	Factor
PT – KG V1	4.00
PT – KG V2	4.00
PT – KG V3	4.00
PT – KG H1	4.00
PT – KG H2	4.00
PT – KG H3	4.00

Load combination basis PT-t1:

Load case	Factor
PT – t0	0.94

Load combination basis PT-t2:

Load case	Factor
PT – t0	0.84

3.14 LOAD COMBINATIONS

Verification of load capacity shall be carried out for several limit states as detailed in this section.

Fatigue Limit State:
The risk of fatigue according to the partial factor method is checked using equation 6.69 provided in document SS-EN 1992-1-1.

Other Limit States:
For other limit states, section 6.4.3 of EN-1990 is applied.

3.14.1 Ultimate Limit States (ULS)

When checking the ultimate limit state, the load factors vary depending on the type of failure as detailed below:

STR: Verification of structural bearing capacity

GEO: Verification of geotechnical bearing capacity

For checking the ultimate limit state, TRVNFRA-00227 section 7.1.6.3 specifies requirements for load combinations as follows.

Design Method D2 (Set B):
Design Method D2 (Set B) according to TSFS 2018:57 Table 4.4 shall be applied for the structural bearing capacity of the construction (STR; SK 3).

Design Method is defined according to EN-1990 equations 6.10a and 6.10b as detailed below.

$$E_{Sd}^{10a} = \sum_{j\geq 1} \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot \psi_{0,1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-A} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

$$E_{sd}^{10b} = \sum_{j\geq 1} \xi_j \cdot \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-B} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Equation 6.10a refers to the (ULS-A) case where the permanent loads are dominant, usually during the construction phase.

Equation 6.10b refers to the (ULS-B) case where the variable loads are dominant.
Design method 2 (set B) according to TSFS 2018:57 table 4.4 shall be applied for the structural capacity (STR; SK3).

A1 (construction loads)
All load factors are greater than set C.

A2 (geotechnical loads)

- Load coefficient earth pressure:
  $\psi\gamma_{ULS-A} = \gamma_d \cdot 1.35 \cdot \eta_{sup.G} = 1.0 \cdot 0.89 \cdot 1.35 \cdot 1.1 = 1.49$ ← dimensioning
  $\psi\gamma_{ULS-B} = \gamma_d \cdot 0.89 \cdot 1.35 \cdot \eta_{sup.G} = 1.0 \cdot 0.89 \cdot 1.35 \cdot 1.1 = 1.33$

- Load coefficient surcharge:
  $\psi\gamma_{ULS-A} = \gamma_d \cdot \psi_0 \cdot 1.50 = 1.0 \cdot 0.75 \cdot 1.50 = 1.13$
  $\psi\gamma_{ULS-B} = \gamma_d \cdot 1.50 = 1.0 \cdot 1.50 = 1.50$ ← dimensioning

Design method D3 (set C):

Design method D3 (set C) according to TSFS 2018:57 table 4.5 shall be applied for determining geotechnical bearing capacity (GEO; SK 2).

The design method is defined according to EN-1990 equation 6.10a and 6.10b as presented below.

$$E_{Sd}^{10a} = \sum_{j\geq 1} \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot \psi_{0,1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-GA} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

$$E_{sd}^{10b} = \sum_{j\geq 1} \xi_j \cdot \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-GB} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Equation 6.10a refers to the (ULS-A) case where the permanent loads are dominant, usually during the construction phase.

Equation 6.10b (ULS-B) refers to the case where the variable loads are dominant.

Design method 3 (set C) according to TSFS 2018:57 table 4.5 shall be applied for determining geotechnical bearing capacity (GEO).

A1 (construction loads)
All load factors are less than set B.

A2 (geotechnical loads)

- Load coefficient earth pressure: $\psi\gamma_{jord} = \gamma_d \cdot 1.1 \cdot \eta_{sup.G} = 0.91 \cdot 1.1 \cdot 1.1 = 1.10$

- Load coefficient surcharge: $\psi\gamma_{\text{över}} = \gamma_d \cdot 1.40 = 0.91 \cdot 1.40 = 1.27$

Simplified Design Method ULS:

To limit the number of load combinations, design method D2 (STR) is also applied for checking geotechnical bearing capacity (GEO). This is done by adjusting load coefficients associated with the geotechnical loads.
When applying the geotechnical loads, the earth pressure coefficient corresponding to D2 is applied.

Check load coefficients associated with the geotechnical loads

$$K_o(D2) = 1 - sin(\varphi_d) = 1 - sin45^\circ = 0.29$$

$$K_o(D3) = 1 - sin(\varphi_d) = 1 - sin38^\circ = 0.39$$

Earth pressure→ 1.48[1.)] ·K_0(D2) = 0.43 ≡ 1.10 · K_0(D3) = 0.43 i.e. OK!

Surcharge → 1.71[2.)] ·K_0(D2) = 0.43 ≡ 1.27 · K_0(D3) = 0.50 i.e. OK!

Footnotes

[1.)] Last coefficient $\psi\gamma_{ULS}$ = 1.48 is applied instead of 1.33.

[2.)] Last coefficient $\psi\gamma_{ULS}$ = 1.71 is applied instead of 1.50.

Permanent loads:

Nr	Load		$\psi\gamma_{ULS-A}$	$\psi\gamma_{ULS-B}$		$\psi\gamma_{ULS}$
1	Egentyngd	max	1.35	1.20		1.20
		min	1.00	1.00		1.00
2	Beläggning	max	1.49	1.33		1.33
		min	0.90	0.90		0.90
3	Överfyllnad	max	1.49	1.33		1.33
		min	0.90	0.90		0.90
4	Jordtryck	max	1.49	1.33	⇒	1.48 [1.)]
		min	0.90	0.90		0.90
5	Vattentryck	max	1.35	1.09		1.09
		min	1.00	1.00		1.00
6	Stödförskjutning	max	1.35	1.20		1.20
		min	1.00	1.00		1.00
7	Krympning	max	1.35	1.20		1.20
		min	1.00	1.00		1.00
8	Spännkraft	max	1.35	1.35		1.35
		min	1.00	1.00		1.00

Footnote:

[1.)] Load coefficient according to page A3:172 is applied.

Remark

Equation ULS-B is considered dominant; thus ULS-A is not considered.

Variable loads:

Nr	Load	ψγ ULS-A	ψγ ULS-B		ψγ ULS
	Lastmodell LM 1 :				
9	Boggiesystem	1.13	1.03/1.50		1.03/1.50
10	Utbredd last	0.60	0.60/1.50		0.60/1.50
11	Bromskraft	0.84	0.84/1.13		0.84/1.13
12	Sidokraft	0.84	0.84/1.13		0.84/1.13
13	Centrifugalkraft	0.84	0.84/1.13		0.84/1.13
	Lastmodell LM 2 :				
14	Enstaka axellast	0	0/1.50		0/1.50
	Typfordon EG A/B :				
15	Typfordon EG A/B	1.13	1.13/1.50		1.13/1.50
16	Bromskraft	0.84	0.84/1.13		0.84/1.13
17	Sidokraft	0.84	0.84/1.13		0.84/1.13
18	Centrifugalkraft	0.84	0.84/1.13	⇒	0.84/1.13
19	Temperatur	0.90	0.90/1.50		0.90/1.50
	Vindlaster:				
20	Vindlast mot bro	0.45	0.45/1.50		0.45/1.50
21	Vindlast mot trafik	0.45	0.45/1.50		0.45/1.50
22	Överlast	1.13	1.13/1.50		1.13/1.71[2.)]

Footnote:

[2.)] Load coefficient according to page A3:172 is applied.

Remark

Equation ULS-B is considered dominant; thus ULS-A is not considered.

Load combination smart *ULS-PERM :*

Load case	Permanent factor	Variable factor
EGEN	1.00	0.20
BELÄGG	0.90	0.43
JORD	0.90	0.58
STOD	0	0.30 = (1.20 x 0.37[1.)] x 0.6[2.)])
KRYMP	0	0.30 = (1.20 x 0.37[1.)] x 0.6[2.)])
PT-t0	0.84 (= 0.84·1.00) [3.)]	0.51 (= 1.35 - 0.84) [3.)]

Footnotes:

[1.)] The effect of creep results in reduced stiffness; see page A3:46.

[2.)] The effect of cracking results in reduced stiffness; see page A3:114.

[3.)] Load case pretension varies from PT-t0 to PT-t2 (= 0.84·PT-t0) is applied.

Although the load cases STOD and KRYMP do not need to be considered according to SS-EN 1992-1-1, this is done on the safe side.

Load combination smart *ULS-PERM-0 :*

(Identical to ULS-PERM but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
EGEN	1.00	0.20
BELÄGG	0.90	0.43
JORD	0.90	0.58
STOD	0	0.30
KRYMP	0	0.30

Load combination smart *ULS-VAR :*
(Load cases to consider : 6 / Variable load cases : 1)

Load case	Permanent factor	Variable factor
TRAFIK	1.03	0.47
BROMS	0.84	0.29
SIDO	0.84	0.29
TEMP-ULS	0.90	0.60
OVER	1.13	0.58
VIND	0.45	1.05

Load combination smart *ULS :*

Load case	Permanent factor	Variable factor
ULS-PERM	1	0
ULS-VAR	0	1

Load combination smart *ULS-0 :*
(Is identical to ULS but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
ULS-PERM-0	1	0
ULS-VAR	0	1

3.14.2 Service limit state (SLS)

The service limit state is divided into 3 load combinations based on their duration. The load combinations are presented below.

Load combination	Duration
SLS:K	Characteristic
SLS:F	Frequent
SLS:Q	Quasi-permanent

Load Combination SLS:K according to EN 1990 equation 6.14b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + Q_{k.1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{SLS.K} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Load Combination SLS:F according to EN 1990 equation 6.15b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + \psi_1 \cdot Q_{k.1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} = \psi\gamma_{SLS.2} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Load Combination SLS:Q according to EN 1990 equation 6.16b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + \sum_{i>0} \psi_{2,i} \cdot Q_{k,i} = \psi\gamma_{SLS.Q} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

When designing, load coefficients according to equations 6.14a, 6.15b, and 6.16b are applied. Refer to the derivation in Appendix 2.

Adjustment of load coefficients for geotechnical loads:

$K_o(D2) = 1 - sin(\varphi_d) = 1 - sin45^\circ = 0.29$

$K_o(D3) = 1 - sin(\varphi_d) = 1 - sin38^\circ = 0.39$

LC	Earth pressue
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.1 = \frac{0.39}{0.29} \cdot 1.1 = 1.48$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 1.1 = \frac{0.39}{0.29} \cdot 1.1 = 1.48$
SLS-Q	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.0 = 1.34$

LC	Temparature
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.00 = 1.34$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 0.60 = \frac{0.39}{0.29} \cdot 0.60 = 0.81$
SLS-Q	0

LC	Surcharge
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.00 = 1.34$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 0.75 = \frac{0.39}{0.29} \cdot 0.75 = 1.01$
SLS-Q	0

Permanent loads:

Nr	Load		$\psi\gamma_{SLS-K}$	$\psi\gamma_{SLS-F}$	$\psi\gamma_{SLS-Q}$
1	Egentyngd	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
2	Beläggning	max	1.10	1.10	1.00
		min	0.90	0.90	1.00
3	Överfyllnad	max	1.10	1.10	1.00
		min	0.90	0.90	1.00
4	Jordtryck	max	1.48[3.)]	1.48[3.)]	1.34[3.)]
		min	0.90	0.90	1.00
5	Vattentryck	max	1.00	1.00	1,00
		min	1.00	1.00	1.00
6	Stödförskjutning	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
7	Krympning	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
8	Spännkraft	max	1.00	1.00	1.00
		min	1.00	1.00	1.00

Footnote:
[3.)] Load coefficient page A3:175 is applied.

Variable loads:

Nr	Load	$\psi\gamma_{SLS-K}$	$\psi\gamma_{SLS-F}$	$\psi\gamma_{SLS-Q}$
	Lastmodell LM 1 :			
9	Boggiesystem	0.75/1.00	0/0.75	0
10	Utbredd last	0.40/1.00	0/0.40	0
11	Bromskraft	0.56/0.75	0/0.56	0
12	Sidokraft	0.56/0.75	0/0.56	0
13	Centrifugalkraft	0.56/0.75	0/0.56	0
	Lastmodell LM 2 :			
14	Enstaka axellast	0.75/1.00	0/0.75	0
	Typfordon EG A/B :			
15	Typfordon EG A/B	0.75/1.00	0/0.75	0
16	Bromskraft	0.56/0.75	0/0.56	0
17	Sidokraft	0.56/0.75	0/0.56	0
18	Centrifugalkraft	0.56/0.75	0/0.56	0
19	Temperatur	0.60/1.00	0.50/0.60	0.50
	Vindlaster:			
20	Vindlast mot bro	0.30/1.00	0/0.30	0
21	Vindlast mot trafik	0.30/1.00	0/0.30	0
22	Överlast	0.75/1.34[4.)]	0/1.01[4.)]	0

Footnote:

[4.)] Load coefficients according to page A3:175 is applied.

Load combination smart *SLS-PERM :*

Loadcase	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	0.90	0.20
JORD	0.90	0.58
STOD	0	0.22 = (1.0 x 0.37[1.)] x 0.6[2.)])
KRYMP	0	0.22 = (1.0 x 0.37[1).] x 0.6[2.)])
PT-t0	0.84[3.)]	0.16[3.)]

Footnotes:

1.) The effect of creep results in reduced stiffness; see page A3:46

2.) The effect of cracking results in reduced stiffness; see page A3:114

3.) Load case pretension varies from PT-t0 to PT-t2 (= 0.84·PT-t0) is applied.

Load combination smart *SLS-PERM-0 :*

(Identical to SLS-PERM but does not contain load case PT-t0)

Loadcase	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	0.90	0.20
JORD	0.90	0.58
STOD	0	0.22
KRYMP	0	0.22

Load combination smart *SLS-K-VAR :*

(Load cases to consider : 6 / Variable load cases : 1)

Load case	Permanent factor	Variable factor
TRAFIK	0.75	0.25
BROMS	0.56	0.19
SIDO	0.56	0.19
TEMP	0.60	0.40
OVER	0.75	0.59
VIND	0.30	0.70

Load combination smart *SLS-F-VAR :*

Load case	Permanent factor	Variable factor
TRAFIK	0	0.75
BROMS	0	0.56
SIDO	0	0.56
TEMP	0	0.60
OVER	0	0.95
VIND	0	0.30

Load combination smart *SLS-K :*

Load case	Permanent factor	Variable factor
SLS-PERM	1	0
SLS-K-VAR	0	1

Load combination smart *SLS-K0 :*
(Is identical to SLS-K but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
SLS-PERM-0	1	0
SLS-K-VAR	0	1

Load combination smart *SLS-F :*
(SLS-F0 is identical but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
SLS-PERM	1	0
SLS-F-VAR	0	1

Load combination smart *SLS-F0 :*
(Is identical SLS-F0 but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
SLS-PERM-0	1	0
SLS-F-VAR	0	1

Load combination smart *SLS-Q :*
(SLS-Q0 is identical but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	1.00	0.20
JORD	1.00	0.34
STOD	0	0.22 = (1.0 x 0.37[1.)] x 0.6[2.)])
KRYMP	0	0.22 = (1.0 x 0.37[1).] x 0.6[2.)])
PT-t0	0.84[3.)]	0
TEMP-SLS Q	0	0.50

Footnotes:

[1.)] The effect of creep results in reduced stiffness; see page A3:46

[2.)] The effect of cracking results in reduced stiffness; see page A3:114

[3.)] Load case pretension PT-t2 (= 0.84·PT-t0) is assumed.

3.14.3 Accidental load combination

The accidental load case is also designated as exceptional load combination EXC according to SS-EN 1990 section 6.4.3.3 equation 6.11a as shown below.
The accidental load case is denoted as A_d and consists of cable loss or impact load.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + P + A_d + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} = \cdots$$

$$\psi\gamma_{EXC} \cdot \left(\sum_{j\geq 1} G_{k,j} + P + A_d + \sum_{i\geq 1} Q_{k,j}\right)$$

3.14.4 Fatigue load combination

Fatigue is considered according to SS EN 1992-1-1, 6.8.4 and 6.8.6, and SS EN 1992-2, 6.8 and Appendix NN.

The risk of fatigue is checked using a simplified method, denoted as the λ-method.
Load combination according to equation SS-EN 1992-1-1 section 6.8.3 equation 6.69.

In this load combination, the traffic load is considered to consist of UTM, whereby other traffic loads are excluded.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} + Q_{fat} = \psi\gamma_{UTM} \cdot (\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} Q_{k,i} + Q_{fat})$$

Permanent loads:

Nr	Load		$\psi\gamma_{UTM}$
1	Egentyngd	max	1.00
		min	1.00
2	Beläggning	max	1.10
		min	0.90
3	Överfyllnad	max	1.10
		min	0.90
4	Jordtryck	max	1.48
		min	0.90
5	Vattentryck	max	1.00
		min	1.00
6	Stödförskjutning	max	1.00
		min	1.00
7	Krympning	max	1.00
		min	1.00
8	Spännkraft	max	1.00
		min	1.00

Variable loads:

Nr	Load	ψγ$_{UTM}$
	Lastmodell LM 1 :	
9	Boggiesystem	-
10	Utbredd last	-
11	Bromskraft	-
12	Sidokraft	-
13	Centrifugalkraft	-
	Lastmodell LM 2 :	
14	Enstaka axellast	-
	Typfordon EG A/B :	
15	Typfordon EG A/B	-
16	Bromskraft	-
17	Sidokraft	-
18	Centrifugalkraft	-
19	Temperatur	0.60
	Vindlaster:	
20	Vindlast mot bro	0.30
21	Vindlast mot trafik	0.30
22	Överlast	1.01
23	UTM3	1.00

Load combination smart FAT :

(FAT-0 is identical but does not contain load case PT-t0)

Load case	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	1.00	0
JORD	1.48	0
STOD	-	-
KRYMP	-	-
PT-t0	0.84	-
VIND	-	-
UTM	-	1.00
OVER	-	-
TEMP	-	-

Load cases BELÄGG, STOD and KRYMP are not fatigue loads, thus load coefficient 1.0 is applied.

Load cases pretension is not a fatigue loads, thus load coefficient lowest load value of value is assumed PT-t2 (= 0.84·PT-t0) is applied.

Load case JORD is not a fatigue load, thus load coefficient highest load coefficient is applied.

Load cases TEMP, VIND and OVER are not fatigue loads, thus load is not considered.

During verification STR, the load case TEMP can be neglected according to SS-EN 1992-1-1 section 2.3.1.2(2).